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EVALUATION OF APL OTEC 10/20 MWE PILOT PLANTSHIP

Final Report

December 1979

Work Performed Under Contract No. AC02-78ET21002

Det Norske Veritas
Oslo, Norway

U.S. Department of Energy

Solar Energy

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EVALUATION OF APL OTEC 10/20 MWE
PILOT PLANTSHIP

FINAL REPORT

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Oslo, Norway

Prepared Under Contract to
VSE CORPORATION
2550 Huntington Avenue
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CONTENT

EXECUTIVE SUMMARY

SUMMARY AND CONCLUSIONS

1. INTRODUCTION AND SCOPE
2. GENERAL DESCRIPTION OF CONCEPT
3. DESIGN PRINCIPLES AND CODES
 - 3.1 General
 - 3.2 Design figures hull structure
 - 3.3 Definition of loading conditions
 - 3.4 Comparison with other codes
 - 3.5 Coldwater pipe
 - 3.6 Summary
4. MATERIAL SPECIFICATION AND SUITABILITY
 - 4.1 General
 - 4.2 Concrete Quality Hull Structure
 - 4.3 Concrete Quality CWP
 - 4.4 Cement
 - 4.5 Reinforcing Steel
 - 4.6 Prestressing Materials
 - 4.7 Prestressing Grout
 - 4.8 Neoprene Pads
 - 4.9 Teflon for Sliding Surfaces
 - 4.10 Stainless Steel
 - 4.11 Corrosion of embedded steel items
 - 4.12 Quality Assurance Program
5. ENVIRONMENTAL LOADINGS AND RESPONSES
 - 5.1 Environmental conditions
 - 5.2 Hull Structure loadings and Responses
 - 5.2.1 Longitudinal bending and Shear
 - 5.2.2 Water pressure on Panels
 - 5.2.3 Seakeeping

- 5.2.4 Hydrostatic Stability
- 5.2.5 Damage Stability
- 5.3 Coldwater Pipe loadings and Responses
 - 5.3.1 Method of Analysis
 - 5.3.2 Lateral Loads
 - 5.3.3 Axial loads
 - 5.3.4 Dynamic Stability

6. ANALYSIS AND DESIGN

- 6.1 Introduction
- 6.2 Hull Structure
 - 6.2.1 Method of Analysis
 - 6.2.2 Load Coefficients
 - 6.2.3 Serviceability Conditions
 - 6.2.4 Capacity Check
 - 6.2.5 Prestressing Scheme
 - 6.2.6 Reinforcement
 - 6.2.7 Cover to reinforcement
 - 6.2.8 Miscellaneous Effects
- 6.3 Design of CWP
 - 6.3.1 Introduction
 - 6.3.2 Extent of present design
 - 6.3.3 Design of CWP segments
 - 6.3.4 Minimum reinforcement
 - 6.3.5 Cover to reinforcement
 - 6.3.6 Construction details
 - 6.3.7 Typical hinge between segments
 - 6.3.8 Top hinge

7. CONSTRUCTION AND INSTALLATION

- 7.1 Construction site
- 7.2 Construction Methodology of Platform
- 7.3 Platform Deployment
- 7.4 CWP Construction
- 7.5 Hinge Materials/Assemblies
- 7.6 CWP Deployment
- 7.7 Retrieval

8. INSPECTION AND MAINTENANCE

- 8.1 General
- 8.2 Hull Structure
- 8.3 Cold water pipe

List of References

Appendix A : Submitted documentation by APL

Appendix B : Extract from DnV "Ships' Load & Strength Manual, section 4, shipmotion and local loads". pages 27-34.

Appendix C : "Offshore Inservice Inspection and Monitoring - A Basis for Efficient Operation and Maintenance" Veritas Conferance 27th - 28th Nov. 1979, Høvik, Oslo. Dr.ing. Svein Fjeld

EXECUTIVE SUMMARY

Under contract with V.S.E. Corporation (formerly Value Engineering) Det norske Veritas has carried out an independent evaluation of the APL Pilot Plant Concept for an OTEC 10/20 MW_e Power Plant.

The object of this feasibility study has been to verify the structural feasibility of concrete hull and cold water pipe (CWP).

This has been done through an evaluation of the following items:

- design philosophy and criteria
- material specifications and testing
- environmental conditions and loads
- structural analyses and design
- constructability and deployment
- inspection & maintenance

The study has been concerned with mainly the use of new technique; new materials; and new applications of recognized techniques in the project. Details which can be solved easily by recognized methods have not been considered.

The study is based on the documentation referenced in appendix A, on DnV's independent investigations and on a meeting with the

designers.

The APL proposal is found to be a feasible and sound conceptual design, meeting all the basic requirements of an OTEC plant. The application of concrete in the hull and in the CWP seems to offer great advantages, mainly related to durability, inspection and maintenance.

A few points, which we believe should be investigated further prior to the final design, have been discussed in the body of the report and are briefly summarized below.

Lightweight concrete

The requirement for low weight, high strength and high durability of the CWP has led to the development of a special type of lightweight concrete. Tests have been carried out and a future test program is specified to demonstrate compliance with these requirements. This report suggests further testing, particularly regarding creep, permeability and fatigue strength. Experience with offshore concrete has demonstrated the importance of these properties for a proper design.

Response of CWP, Mathieu effect

The response of the CWP has been studied by the Paulding ROTEC program. It seems that the effect of the vertical oscillation of

pipe (caused by the heave motion of the ship) which may cause increased transverse amplitude of the pipe, has not been included in this analysis. It is recommended therefore that the significance of this effect be studied prior to the final design.

CWP uplift

The factors governing a possible lifting of the CWP from its seat on the hull, will be the heave acceleration of the barge and the submerged unitweight of the CWP material. Present information indicates that the conditions at the Puerto Rico site are critical. Increased dimensions or weight of the barge and/or increased unitweight of lightweight concrete can solve this problem.

Design of CWP

The CWP design calls for a delicate balance of wall thickness to reduce weight, with high strength and durability to ensure reliable performance during the intended lifetime of the plant. The proposed wall thickness has been reduced to the minimum. It appears that the reinforcement in the circumferential direction is not enough. We believe that a careful design which includes all loads causing hoop stresses on the pipe, and proper considerations of the minimum amounts of reinforcement necessary to

properly control and distribute the cracks, will result in an increase in the hoop reinforcement.

Practical experience and theoretical work in Europe have demonstrated that sharp corners such as those on the CWP elements at the bell and spigot ends increase the corrosion of reinforcement and spalling of the concrete. This corrosion risk should be seriously considered when detailing the CWP-elements.

Great care must be exerted to insure that there is no "slop" in the joints between CWP segments, to avoid pounding of those joints.

Response of Hull

The response calculation of the hull structure is not anticipated to represent any significant problem as this is in line with common ship practice. However, the loading conditions given should be subject to further parametric studies. Other combinations of wave heights and wave periods may give higher loadings than those calculated in present design.

Panel Pressure on Hullside

The APL design of dynamic panel pressure is based upon semi-empirical methods derived for steel ships which is related to a certain stress level. For a concrete barge the situation will

be different and should be based on the actual extreme conditions. The figures calculated in this report, therefore, are higher than those used by APL. The significance of this is reduced because APL has used higher load coefficients than are used in this report.

Conclusion

We stress that none of the above mentioned items will prevent the realization of the project.

Provided that the investigations suggested here are carried out and interpreted carefully, the APL conceptual design seems to be a sound and valid solution, based on good engineering. It is well suited to be the basis for the detail design of a plant ship of the given size and for the given location.

Assuming a proper detail design, construction and installation, taking advantage of the experience and know-how of previous offshore concrete structures, we feel that the project can be successfully realized.

The lack of experience with and knowledge to the proposed lightweight concrete is a point of uncertainty in this concept. The current test program phase I and the forthcoming phase II testings seems feasible to investigate its suitability for offshore use.

This report discusses the most important properties of lightweight concrete for offshore use and suggests improvements to the test programs related to properties such as permeability, creep and fatigue.

Environmental loadings and Responses

The environmental loadings and the structural response have been studied. It seems to be good agreement with the calculated bending moments and shear forces by APL with the simple checks carried out in this report. We have however noted that a higher extreme wave at Puerto Rico location gives smaller bending moment than the lower extreme wave at the Brazil location. From this we conclude that a parametric study to identify the more unfavourable wave height/period should be carried out. We have further noted that lateral longitudinal bending has not been considered in the study, although the thrusters should exclude beam sea it should at least be included as an accidental load combination.

In the estimation of water pressure on panels it seems that the dynamic contribution has been underestimated. But with the higher load coefficient used by APL on hydrostatic pressure for design of wall panels, the ultimate capacity may not need improvements. However the serviceability conditions may become critical and it is therefore recommended to use the loadings and load coefficients specified by DnV for this analysis.

Another effect which could become a serious problem for the hull structure would be the water inside the CWP being accelerated with the heave of the barge causing high vertical loads acting midship. However with the modified solution for pumps and pipe arrangement in the CWP pool this problem seems solved, provided proper opening area between the CWP and the suction pool to allow for adequate waterflow.

The problems associated with heave acceleration will also determine the actual possibilities for uplift of CWP at the hull connection and the load reversal at element joints. The latest modification of pumps and pipe arrangement should also improve this condition and reduce the possibility of uplift. The criteria for checking uplift will therefore be the specific weight of the lightweight concrete and the heave acceleration of the barge. The APL design indicates heave accelerations for extreme conditions at the Puerto Rico site which will result in uplift of the CWP with the present specifications of lightweight concrete. This is an unacceptable condition for the present barge CWP configuration.

Design of Hull and CWP

The present design of the hull structure seems adequate. Serviceability control, however, of crackwidth has been recommended as additional check. The concrete cover to prestressing ducts at different places seems small and should be subject to further considerations.

For the GWP we believe the proposed design approach should be sufficient with minor adjustments. The main concern is the behaviour of the element joints and if this can be properly investigated with regard to displacement and forces. The phase II test models might give valuable information on this.

In addition we are somewhat concerned about the structural stability of the thin walled elements. Proper investigation of the element hoop stresses must be included in the final analysis. Similarly the construction method of thin walled elements (4.5") over height of 25 or 50 ft. needs to be carefully examined. Again the proposed model tests may give valuable information.

Regarding the hinge connection between the hull and the CWP we believe the principle function of this to be relatively simple. Our main concern for this hinge is more related to the question of construction, maintenance and possible repair. A gradual abrasion of the teflon sliding surface will increase the friction of this hinge causing higher stresses in the pipe elements. We find no allowance for this in the present design.

Construction and Installation

The proposed APL program for construction and installation of the hull structure and the coldwater pipe elements represents a comprehensive and thoroughly prepared document verifying the feasibility of this work. The report therefore include only minor remarks to this proposal.

Inspection and Maintenance

The APL concept includes only limited discussion of this subject. Although experience on offshore concrete structures indicates a minimum of maintenance work, in-service inspection should be prepared. This report therefore discusses in general terms the type and extent of such inspection work.

Conclusion

In general the APL Pilot Plant Concept design represents a technically good and feasible solution for a plant ship of the given size. It further demonstrates that concrete will be a suitable material for construction of this plant, although the lightweight concrete for CWP use needs further development and testing.

The greatest uncertainty is represented in the design of the CWP, both regarding the response of the pipe and the final properties of the lightweight concrete. But the proposed development program of future investigations and testings should clarify the present uncertainties.

We believe that the present uncertainties can be clarified and that in the final design due account will be given to the effects mentioned in this report, resulting in a feasible solution of the OTEC pilot plant concept.

1. INTRODUCTION AND SCOPE

Upon request from V.S.E. Corporation (formerly Value Engineering) job no. 0015-502 B Det norske Veritas has carried out an independent evaluation of the Applied Physics Laboratories (APL) preliminary design concept for an OTEC 10/20 MW Pilot Plant Ship.

The main purpose of the study has been to evaluate on an independent basis the structural integrity and safety of the concept hull structure and the cold water pipe (CWP) during its intended operation. The study has been based upon the evaluation of submitted documentation from APL, a complete list of these documents are given in Appendix A. The main items in the study can be summarized in the following:

- Check that the general design principles, and reference codes are adequate and in line with present state of the art
- Evaluating the suitability of the material specified for its intended purpose
- Evaluating the environmental conditions, loadings and structural response for the hull and CWP
- Evaluating the applied design method and analysis used, checking the design according to the specified codes
- Evaluating the feasibility of the proposed construction and installation method
- Evaluating of the suitability of the proposed inspection and maintenance system and suggest necessary modifications
- Evaluating the suitability of the given positioning system, checking stability during intact and damaged conditions and study the given seakeeping conditions.

The study concentrates on the essential items which contributes to the structural strength and durability of this concept. Suggestions for possible modifications and improvements will be made, and recommendations for necessary further studies and testings will be given.

A detailed design check will not be included at the present stage, and the evaluation of the power and process systems are considered outside this scope of work.

2. GENERAL DESCRIPTION OF CONCEPT

The OTEC 10/20 MW Pilot Plant Concept is developed by Applied Physics Laboratories at John Hopkins University under a contract with U.S. Department of Energy. The design of the concrete structure is carried out by ABAM Consulting Engineers.

The concept consist of a prestressed concrete barge (hull structure) with a cold water pipe of lightweight concrete.

The barge has the following principal dimensions:

Length	378 feet	(115 m)
Beam	121 feet	(37 m)
Depth	89 feet	(27 m)
Draft (design)	65 feet	(20 m)
Displacement	67.900 L.T.	
Hull weight	46.500 L.T.	

Figure 2-1, illustrates the general arrangement of the barge and the CWP.

The cold water pipe (CWP) 30 ft (9.2 m) in diameter extends 3000 ft (1000 m) below the barge. It consist of 60 elements each 50 ft long joined together with a breech-type locking system. The joints are built up with elastomeric bearing pads allowing some rotation which give hinge effect at the joints. The CWP has a ball joint type connection (Gimble bearing) to the hull allowing for free rotation.

The CWP will be made of lightweight concrete developed for this concept with a saturated density of 85 lbs/ft³ (1,38 ton/m³). Typical details of the CWP elements are given on fig. 2-2, 2-3.

The hull structure has space for installation of four 5MW_e replaceable power modules of which two giving a total of 10MW_e , will be installed initially.

The structure will be designed for two alternative sites one outside Puerto Rico, where it will be moored, and the other northeast of Brazil (ATL-1), where the plant will be cruising to utilize the optimum temperature difference. The plant is designed to operate under normal conditions but will be shut down during hurricanes and extreme conditions, in which the hull structure will be deballasted to reduced draft.

Figure 2-1

COLD WATER PIPE SEGMENT

SECTION THRU PIPE JOINT

detail slightly modified

Figure 2-3

3. DESIGN PRINCIPLES AND CODES

3.1 General

The concrete hull and coldwater pipe design is carried out by ABAM Engineers, Inc. The company has developed and constructed an ARCO concrete barge for LPG storage facilities of similar dimensions earlier on and should therefore have useful experience with offshore concrete structures. The selected design principle is also in line with the previous job.

The basic design loadings are according to ABS rules for steelship, however, comparisons to other rules as DnV-steelship rules and NKK rules for concrete barges are made. In addition checks are made to verify that the ABS-rules give results to the conservative side with regard to overall bending shear and torsion. Regarding plate bending due to hydrostatic loadings this has been developed as an average from several rules. For details on the loadings, see chapter 5.

The design of the concrete hull structure and CWP will be based upon the ACI-code 318-77 "Building code requirements for reinforced concrete", which is an acceptable code for offshore use.

3.2 Design figures Hull Structure

The following design figures have been chosen for the hull structure /1/:

- 1) Longitudinal bending (Hog/Sag)

Normal Service Conditions

$$M_{\text{cracking}}/M_{\text{actual}} = 2.0$$

$$M_{\text{ultimate}}/M_{\text{actual}} = 2.6$$

$$f_{\text{comp. allowable}} = 0.45 f_c'$$

$$f_{\text{tens. allowable}} = 0$$

$$f_{\text{cracking}} = 7.5/f_c'$$

100-year Storm

$$M_{\text{ultimate}}/M_{\text{actual}} = 1.3$$

ii) Hydrostatic loading (Platebending)

Normal Service Conditions

$$M_{\text{ultimate}}/M_{\text{actual}} = 1.7$$

$$f_{\text{comp. allowable}} = 0.45 f_c'$$

$$f_{\text{tens. allowable}} = 6\sqrt{f_c'}$$

Damaged Conditions

$$M_{\text{ultimate}}/M_{\text{actual}} = 1.3$$

f_c' = specified compressive stress of concrete, psi:

The above specified design principle and loading coefficients are in line with the current philosophy used for steel and concrete design for offshore structures, and also adopted by many codes worldwide.

3.3 Definition of loading conditions

To analyse the proposed design principles we need to define the actual loading conditions for which a concrete structure needs to be designed this can be stated as follows.

3.3.1 Normal Operating Conditions

This must include the conditions for which the owner can safely operate the energy production on the plant, and must be decided according to the sensitivity of his equipment and acceptable shut-down periods.

3.3.2 Ordinary Condition

In addition to normal operating condition which gives criteria for the owners operation of the structure an ordinary loading condition related to a given level of loading is usually given in the different codes.

This condition may not necessarily be the same as the normal operating condition given above.

The purpose of introducing an ordinary loading condition is to obtain a more even level of safety for members subjected to combined permanent, live and environmental loads. The DnV rules /2/ defines this loadings as follows:

$$\text{ULS} : 1.3 P + 1.3 L + 0,7 E$$

$$\text{SLS} : 1.0 P + 1.0 L + 0,5 E$$

where P - permanent loads, L - live loads and
E - environmental loads (100 - year recurrence period).

The main purpose of the serviceability control will be a control of crackwidth. Steel stress limitation may be used as an alternative to crackwidth control.

3.3.3 Extreme Conditions (100-year Storm)

The extreme condition cover the most probable largest environmental load with a recurrence period of 100 years.

The structure should be designed to sustain this load with an adequate degree of safety. In addition to adequate strength the structure should also fulfill certain serviceability criteria to ensure that normal operation of the structure can take place afterwards.

3.3.4 Accidental Conditions

The design accidents normally include two conditions.

- an accidental load may cause local damage to the structure but a major disaster may be avoided. Criteria for acceptable local damage to be decided upon for each type of accidental loadings to occur.
- prior to the repair of the damage the structure should have sufficient strength to withstand environmental conditions for a recurrence period of normally one year.

3.4 Comparison with other codes

To get an impression of the proposed design level of the OTEC structure we will make a comparison with other codes used in offshore design, f.inst. DnV-Rules for Offshore Structure 1977 /2/, can be used.

3.4.1 Ordinary Conditions/Extreme Conditions

The DnV rules defines the ordinary environmental loadings including load coefficients as 70 % of the extreme conditions.

The figures given by APL for the two sites in question yield the following ratio for longitudinal bending of normal service conditions, which is the APL definition, to extreme conditions.

Brazil (ATL)-1):

$$\text{Hogging moment:} = \frac{240\,000}{330\,000} = 0.73$$

$$\text{Sagging moment:} = \frac{220\,000}{330\,000} = 0.67$$

$$\text{Puerto Rico:} = \frac{240\,000}{276\,000} = 0.87$$

Stillwater bending moment is here considered as a permanent load, and not included in the above figures.

This indicates good agreement between the APL design and the DnV rules when relative figures are concerned but do not include considerations of the actual magnitude of the loadings, which is covered in chapter 5.

3.4.2 Ordinary Conditions

As above, stillwater bending is considered as permanent load.

ULS Design loads

$$\text{APL:} \quad 396.000 \times 2,6 = 1.029.000 \text{ LT feet}$$

$$\text{DnV:} \quad 156.000 \times 1.3 + 0.7 \times 330\,000 = 434.000 \text{ " "}$$

If such comparison can be considered as representative the ratio between APL and DnV becomes 2,37. Material factors specified by DnV and usage factors by ACI may slightly alter this, but nevertheless the figures indicate conservatism at this stage of the APL design. On the other hand DnV - rules /2/ assumes that local effects are included and this complicates the comparison.

Serviceability loadings

$$\text{APL :} \quad 396.000 \times 1.0 = 396\,000 \text{ LT feet}$$

$$\text{DnV :} \quad 156.000 \times 1.0 + 0.5 \times 330\,000 = 321\,000 \text{ " "}$$

$$\text{Ratio APL/DnV} = 1,23$$

The check criterias for the two cases are somewhat different, but no membrane tension i.e. $f_{\text{tens. allow}} = 0$ indicates at least the same level as given in DnV rules.

For combination with local plate bending the serviceability criteria by limiting concrete stresses in stage I design (i.e. no cracking) may not for all conditions give the same limitations in steel reinforcement stresses, i.e. crackwidth, as stated in DnV rules. Additional check should therefore be included to ensure acceptable steel stresses or control of crackwidth.

3.4.3 Extreme Conditions

ULS loadings:

$$\text{APL : } 1.3 \times 486.000 = 631.000 \text{ LT feet}$$

$$\text{DnV : } 1.0 \times 156.000 + 1.3 \times 330.000 = 585.000 - " -$$

This indicates 7 % greater ultimate loadings by APL compared with DnV rules.

Serviceability

Loadings are unfactorized and consequently unchanged, however, the steel reinforcement limitation of $0.8 \times f_y$ is not necessarily covered by APL criterias, as no check is included here. As stated above this is a check to ensure normal use of the structure after an extreme condition. Such a check is therefore highly recommended.

3.4.4 Local Plate loadings

Local plate loading should be combined with the overall loadings for the capacity and serviceability check of all members/sections in question.

Both normal operating and extreme conditions should be considered and an appropriate load factor should be used at both the static and the dynamic portions on the loads. For details of the actual dynamic part of this loading we refer to chapter 5.2.2 on environmental loadings, in which the comparison between APL and DnV figures have been included.

3.4.5 Accidental conditions

The only accidental condition specified is damage condition when checking stability of the barge for certain compartments flooded. Together with an ultimate check of longitudinal bending including a load coefficient of 1.3, this seems to be a reasonably conservative check.

However, no specifications of other accidental events have been given. We will therefore suggest the following cases to be studied,

- Accidental dropped object
- Fire/explosion
- Boat impacts

The philosophy used should be as outlined above.

3.4.6 Other codes

Comparison with other codes could have been used for instance ACI 357 /3/, but this code as well as most of other relevant codes specifies similar criterias as given in DnV codes.

3.5 Coldwater Pipe (CWP)

The proposed design of the CWP is based on forces from analysis of different extreme (100-years) conditions.

Specified ultimate design load is given as /1/:

$U = 1.0 D + 1.3 E_{100}$ which corresponds to DnV rules /2/, however serviceability check for both ordinary and extreme conditions should be included to ensure the durability of the concrete i.e. crackwidth control and fatigue check.

3.6 Summarizing this chapter we conclude as follows:

- Design philosophy used will give an acceptable basis for the design. Some parts indicate conservative figures on ultimate conditions, reductions may therefore be possible if the serviceability criteria will not be governing
- Some additional accidental conditions should be investigated.
- Adopted ACI-318 code criterias are useful but check on steel stresses or alternatively crackwidth control should be included for serviceability condition both during ordinary and extreme conditions.

However, when considering hydrostatic pressure as a live load condition this may give a conservative result. The best solution should be to multiply the different components of the plate loading (static and dynamic part) by their corresponding load factors and then checking both ordinary and extreme conditions.

- Serviceability check for both ordinary and extreme conditions should be included for the CWP.

The comments in this report to the concrete design of hull as well as CWP on several occasions accentuate requirements which are to avoid corrosion of the reinforcement e.g. requirements to permeability and crackwidths. Recent research published in UK and Norway indicates that there are minor correspondance between crackwidths and corrosion (at least for crackwidths less than 0,5 mm) and that rebar corrosion of completely submerged concrete will be minor. These finding are related to North Sea structures and does not cancel the relevance of our comments in this connection for the following reasons:

- The water temperature in the North Sea is normally less than 50° F whereas the temperature surrounding the OTEC plant should be ca. 80° F. Several research programs i.e. /14/ have demonstrated corrosion to increase significantly in this temperature range.
- The optimistic North Sea findings always make reference to a state of controlled cracking. Wide cracks associated with inelastic strain of the rebars are still anticipated to result in corrosion.
- Longitudinal cracks following the rebars - or much worse - following prestressed tendons, is still considered promoting serious corrosion.
- A si qua non for all corrosion protection of rebars and tendons is an adequate cover of durable and almost impermeable concrete.
- The possibility for inspection, maintenance and repair of the CWP is almost excluded. For this reason any possibility for rebar corrosion must be excluded.

4. MATERIAL SPECIFICATION AND SUITABILITY

4.1 General

This chapter covers the material suitability and specifications for structural use of the hull and cold water pipe. Material specifications for heat exchanges and other power and process related structures will not be included in this report.

Both the hull structure and CWP will be made of prestressed concrete. Consequently, this examination concentrates on the concrete material including its different components, reinforcing and prestressing steel.

4.2 Concrete quality hull structure

Minimum 28-days compressive strength of 6000 psi based on standard cylinders are specified, which is in accordance with normal offshore practice. Durability requirements related to permeability of concrete are not given and for offshore structures this is essential to the concrete quality and proper corrosion protection of reinforcement. Durability requirements could either be specified as a permeability number according to standard tests or by limiting the water cement ratio (0.40) and using a minimum cement content of 400 kg/m³. The permeability represent an important parameter when deciding cover to reinforcement.

4.3 Concrete quality CWP

The wish to minimize the CWP selfweight has lead to the development of a special type of lightweight concrete. Furthermore the wall thickness of the pipe must be kept as low as possible to minimize concrete volumes. Needless to say, it is of vital importance to know the material properties in order to obtain a safe design at minimum dimensions.

The material properties of the lightweight concrete have been developed during a series of tests and most of the required properties have been specified and partly tested in phase I of the test series. And with the phase II test series the more comprehensive and time influenced properties will be studied. For the sake of completeness the more important properties of lightweight concrete for structural use offshore are summarized below,

- compressive/tensile strength
- modulus of elasticity
- weight in dry and saturated condition
- creep and shrinkage
- permeability
- fatigue strength

The concrete development program described in /8/ covers the above parameters except permeability and fatigue.

Although most of the other properties have been tested in the phase I program several tests in addition to permeability and fatigue should be extended in the phase II program to obtain reliable properties about this lightweight concrete. It is essential to the whole CWP design that the important properties of this lightweight concrete have been adequately documented prior to the final design. We will therefore make the following remarks about several of the main properties which should be investigated in the next test series:

- Compressive strength: The existing strength parameters yields 28-days compressive cylinder strength of 4000 psi (28 MPa). Although this seems to be in the lower range of what has normally been used for offshore structures it should be considered acceptable, but future testings should however if possible aim at increased strength properties.

- Water absorption: As the weight of the CWP is essential in the design of the pipe and balljoint connection adequate absorption tests under realistic water pressure for long duration should be adopted. However present results yields favourable properties, but extended duration of such testing will be obvious.

- Permeability: The durability of concrete and its ability of proper corrosion protection of reinforcement is highly dependent upon the permeability of hardened concrete. In meeting on 29 October the designer suggests to use epoxy coated reinforcements as an alternative solution to the corrosion problem and assuming that other significant properties as bond strength etc. remains almost unchanged, proper protection of rebars may be possible even with permeable concrete. Recent test results from U.S. ref. /10/ indicates also favourable bond effects for certain epoxy types. With regard to the prestressing cables on the other hand, which represent the most important part of the CWP reinforcement, the risk of corrosion must be kept to a minimum. Consequently low permeability together with other protective methods will be an essential parameter of the lightweight concrete. The permeability properties will also be essential when the minimum concrete cover is to be decided, and is particularly the case if it is necessary to justify figures lower than normal offshore practice.

Tests should therefore be specified to obtain sufficient data about the permeability of this lightweight concrete. Recommended methods for such testings could be as given in the Swedish Code of Practice ref. /11/ or German, Standard DIN 1048.

- Shrinkage and Creep: Shrinkage and creep properties are highly important for determination of minimum reinforcements (cracking) and in particular to determine the actual losses in prestressing strands. Lightweight concrete has normally lower modulus of elasticity and greater creep and shrinkage properties which all together results in greater loss in prestressing strands. And as both the maximum and minimum prestressing will be governing for the design correct creep and shrinkage figures are essential parameter. It is also recommended to include creep tests under repeated loadings in the final test series such to get comparison with creep figures under static loads.

- Fatigue: For parts of the CWP dynamic stresses are of the same order of magnitude as permanent stresses. It is therefore necessary to have some information on fatigue behaviour of the lightweight concrete. We therefore recommend fatigue properties to be established in phase II program. Compression/compression tests on cylinders as proposed may be useful, but in addition submerged compression/tension tests on precracked specimen is recommended to study effect of water pumping in cracks. For test methods reference should be made to OTC paper by Mr. K. Waagaard, ref. /9/.

- Lightweight aggregate: Chemical resistance of the lightweight aggregate has not been given, and this should be specified.

4.4 Cement

The specified portland cement according to ASTM 150-77 represents an acceptable material for offshore use. The C_3A content has been limited to maximum 8 percent, we will however recommend this content to be kept between 5 to 10 percent as both maximum and minimum limitations should be specified. Rapid curing cement for grouting anchor bolts etc. should preferably be selected from a portland cement type. Epoxy resins may be used as an alternative for such cases.

4.5 Reinforcing Steel

Specified steel quality for reinforcing bars of grade 60 and grade 40 according to ASTM A615 are acceptable, and only grade 40 are to be used for welding of reinforcements. Welded Steel Wire Fabric of cold-drawn steel wire welded according to ASTM A185 are also acceptable.

All steel materials are to be delivered with test certificate.

4.6 Prestressing Steel

Prestressing steel of seven wire strand with an ultimate capacity of 270 ksi (1863 MPa) have been proposed. The specifications given for strands, prestressing bars, ducts and anchors seems all to be in accordance with acceptable offshore practice.

Proper information about the relaxation of the prestressing strands should be given by the manufacturer, as this is an important parameter in the determination of losses in prestressing steel.

4.7 Duct Grouting

The most important property of this grout is to have a minimum of bleeding. We will therefore recommend that the requirements specified in DnV Offshore Rules (ref. /2/) should be adopted.

Regarding expansive agents to obtain non-shrinkage grout this should not contain deleterious materials giving risk of brittle failure in prestressing steel.

4.8 Neoprene Pads

Neoprene bearing pads according to ASTM D 2000 should be seawater resistant and have properties suitable for its intended function. The neoprene should preferably be selected from types with documented properties of long term behaviour in submerged conditions.

4.9 Teflon for sliding surface

Teflon is a suitable low friction material for sliding surfaces. However its abrasion resistance is low and if the spherical stainless steel surface in the barge does not remain completely smooth the teflon may gradually be damaged. Marine growth or pitting may cause uneven surface of the stainless steel sliding face.

An alternative low friction material as nylon with higher abrasion resistance should therefore be considered. It seems recommendable to study the effect of varying friction coefficient on the CWP behaviour.

4.10 Stainless Steel

Stainless steel used must be of AISI 316 L type.

4.11 Corrosion of embedded steel items

The corrosion of embedded steel items in concrete structures offshore needs special attention. The possible interaction between the steel items and the reinforcement of the concrete structure must be considered in the cathodic protection design. Otherwise the steel embedment plates should be insulated from the reinforcement, but this is in practice extremely difficult.

If other materials than steel will be used as embedment plates this gives additional considerations regarding galvanic effect problems.

Another aspect for careful consideration for an offshore power plant will be the problem of earthing stray currents with regard to corrosion of steel in general and reinforcing and prestressing steel in particular. This phenomena will be greatly dependant on the availability of direct current.

4.12 Quality Assurance Program

The specified quality assurance program presented has not been studied in any detail at present stage.

The principle philosophy outlined seems to give a sufficient basis for a proper quality control. However, an independent third party check should preferably be included.

5. ENVIRONMENTAL LOADINGS AND RESPONSE

5.1 Environmental conditions

Survival conditions for two sites, Brazil and Puerto Rico, have been defined in sect. 5.2.2.1 and 7.2.1 of ref. /1/:

	<u>Brazil</u>	<u>Puerto Rico</u>
Significant wave height	29 ft	44 ft
Modal wave period ^{x)}	12 sec	13.8 sec

x) = period with highest wave spectral density.

These sea states are intended to represent 100 years extreme storms.

On the basis of the above sea state parameters, the following extreme design waves of sinusoidal form were assumed:

	<u>Brazil</u>	<u>Puerto Rico</u>
Wave height	59 ft	80 ft
Wave length	737 ft	976 ft
Wave period	12 sec	13,8 sec

A thorough evaluation of wave data has been outside the scope of the present review. However, a brief check using results from ref. /4/, based on wave data from /5/, was made. This check, which aimed at a comparison of 100 year extreme wave heights, gave fair agreement with APL's 80 ft wave height at Puerto Rico.

It appeared to be more difficult to produce predictions with satisfactory confidence for the Brazil site, mainly due to the smaller amount of data in the employed wave statistics (approx. 10000 observations). Our results, obtained by the method described in ref. /6/, indicated higher values than the 59 ft given by APL /1/, say wave heights in the range 60 - 80 ft. However, these larger wave heights would be associated with larger wave lengths, thus they would not necessarily produce more severe dynamic loads on the vessel.

The effects with respect to dynamic loads will be further discussed in subsequent sections, along with the adequacy of the selected (assumed) design waves.

5.2 Hull structure loadings and responses

5.2.1 Longitudinal bending and shear

5.2.1.1 A still water bending moment as per the ABS rules has been used in the design:

$$M_{SV} = 156000 \text{ Lt-ft} \quad (\text{sag or hog})$$

As compared with moments found by direct calculation, APL found this to be on the conservative side. E.g. calculated moment at 20 MW_e operating capacity was 125000 Lt.ft(sagging), hogging moment in a survival condition with no ballast 80.600 Ltft.

In the present review the $M_{SV} = 156000 \text{ Lt ft}$ is employed, assuming that it may be either sag or hog.

The above figure is preliminary and needs to be recalculated at the final design stage.

5.2.1.2 Vertical dynamic bending moments given in sect. 5.2.2 of ref. /1/ are to be discussed.

In considering dynamic design loads it is important to realize that classification societies in their rules often give nominal values. These are to be used in conjunction with a set of nominal permissible stresses to yield required scantlings. I.e., the figures for loads are not (necessarily) magnitudes of actual extreme loads. Since such extreme loads are used in the procedure of dimensioning concrete structures, it is necessary to define design loads which are compatible with the employed procedure.

The ABS wave bending moment is not related to any recurrence period or probability level, or rather, this relationship has not been stated explicitly. To prepare for later evaluation of scantlings estimates for loads with 100 years recurrence period are considered,

For comparison with APL values, wave bending moments from ref. /7/ were derived. These are based on results from direct calculations (long term response predictions employing strip theory combined with wave statistics), and give long term extremes at a probability level $Q = 10^{-8}$ (abt. 20 years recurrence period). To obtain 100 year extremes ($Q = 10^{-8.8}$) a factor of $8.8/8.0 = 1.1$ has been employed, yielding.

$$M_W = 98500 \text{ kNm} = 320000 \text{ Lt} - \text{ft}$$

Strictly, ref. /7/ give figures desired on the basis of North Atlantic wave conditions. Other DnV analyses indicate however, that vessels of relatively small length ($L < 120 \text{ m}$ say) may experience about equally high loads in ocean areas with generally less severe wave conditions. A special analysis based on wave statistics for areas

at Brazil or Puerto Rico would give answer to this. Most likely, some reduction would show to be appropriate. The derived value is thus comparable to the quasistatic sinusoidal wave bending moments (100 years extremes) presented in the APL study:

Brazil	330 000 Lt ft
Puerto Rico	276 000 Lt ft x)

x) The lower value for Puerto Rico conditions, represented by the selected higher but longer design wave, should be noted. Actually, the shorter and lower wave might represent a condition also occurring at Puerto Rico. It may thus be a misinterpretation to deduce from the above figures that less severe dynamic bending moments are to be expected at Puerto Rico.

NOTE: The above discussed figures for dynamic wave bending moments do not include any dynamic effect of water in the CWP being accelerated vertically due to ship motions. The relevance of this effect in survival and/or operating conditions should be evaluated.

The latest version of pumping and pipe arrangements (fig. 2 - 2) with a possibility for the water to "escape" from the CWP outlet to the suction pool, and hence helps to avoid water in the CWP to be accelerated due to vertical ship motions. To which degree this goal is achieved, depends upon the design details like opening areas etc. Verification of pump arrangement performance with respect to the above discussed effect through model tests should be considered.

With the present lack of knowledge about the above said effects, the following bending moments are tentatively used in reviewing scantlings:

Still water moment 156 000 Lt ft (sag/hog)

Wave bending moment 330 000 Lt ft (sag/hog)

Total 100 years extreme moment 476 000 Lt ft (sag/hog)

Loads to be applied in conjunction with serviceability control of cracking for ordinary loadings should be derived from the above figures employing relationships pertaining to the design codes used.

5.2.1.3 Lateral dynamic bending

Lateral longitudinal bending has not been considered in the APL study. As an indication, a figure for the lateral bending moment has been derived from a formula given in /7/:

$$M_{W,L} \approx 500\,000 \text{ kNm} = 160\,000 \text{ Lt ft}$$

This is considered a crude estimate, but may be used to evaluate the significance of lateral bending.

Extreme lateral and vertical (dynamic) bending are not simultaneous, the two effects are rather considered to be statistically independent variables.

5.2.1.4 Vertical shear forces

Still water shear forces found by direct calculation /1/ amount 1200 - 1400 Lt at the quarter length sections, and a few hundred tons at midship.

Similarly for the dynamic shear forces:

Region fwd. of AP	Ref. /1/	Ref. /7/
0.2 L - 0.3 L	2800 Lt	2300 Lt
0.4 L - 0.6 L	1830 Lt	1900 Lt
0.7 L - 0.85 L	2800 Lt	2850 Lt

Linear variations towards the ends and in between specified regions can be assumed. The ref. /1/ figure is derived here using a M_w value of 330 000 Lt ft in the ABS' formula (sect. 5.2.2.3 of ref. /1/).

5.2.2 Water pressure on panels

Ref. /1/ uses loads as given in DnV rules for steel ships and NKK rules for prestressed concrete barges. As explained in sect. 5.2.1.2 above, such nominal loads are to be replaced with figures representing actual extreme loads.

Using formulae from "DnV Ships' Load and Strength Manual" ref. /7/ the below values have been derived for dynamic panel pressure. An extract of these formulae are given in Appendix B. Similar methods for deriving panel pressure have also been suggested in DnV Guideline for floating concrete structure ref. /12/.

PANEL/ LOCATION		LEVEL	PRESSURE HEAD (FT)		
			STATIC	DYNAMIC	TOTAL
Side panel	At F.P.	Upper deck	-	29	29
		Still waterline	-	54	54
		Baseline	65	15	80
	Amid-ships	Upper deck	-	6	6
		Still waterline	-	34	34
		Baseline	65	10	75
Upper deck	At F.P.	88 Ft above base	-	21	21
	Amid-ships	88 Ft above base	-	5	5

The dynamic pressures here, which are about twice the values used in /1/, are given a minor allowance for effects due to rolling (abt. 10 % of dynamic value at F.P., abt. 20 % at midships).

A comparison between loads used in /1/ and those given above is shown in figs. 5 - 1 and 5 - 2. Fig. 5 - 3 gives a comparison of the ultimate loads according APL proposal and the above figures factorized according to DnV rules.

5.2.3 Seakeeping

5.2.3.1 Freeboard Determination

The principles used for determining number of exceedances of a preset threshold level (freeboard plus some height) concur with now common practice.

Figure 5-1.

Comparison between panel pressure calculated by APL and DnV.

DECK AND BOTTOM SLAB PRESSURE

AT FORWARD AND AFT

AMIDSHIP

Figure 5-2.

Comparison of deck and bottom slab pressure as calculated by APL and DnV.

APL : $1.7 \times \text{TOT.}$

DnV, ORDINARY: $1.3 \times \text{STATIC} + 0.7 \times \text{DYN.}$

DnV, EXTREME: $1.0 \times \text{STATIC} + 1.3 \times \text{DYN.}$

Figure 5-3.

Comparison of ultimate panel pressure at F.P.

The input parameters employed in this procedure could be commented, however:

- The philosophy behind selection of criterias (acceptable no. of exceedances) is unclear. As compared with ships, the freeboard capacity compares well. The basis for placing requirement on the OTEC is, however, not necessarily the same. This must be based on the operation requirements of the energy producing machinery and its equipments and must be given by the operator.
- The motions of the OTEC are predicted with neglect of possible effects from contained water in the CWP (see sect. 5.2.1.2).
- The rectangular shape of the OTEC barge may give rise to larger relative motions at the F.P. than will be predicted by 2 dimensional strip theory as used here. The reason for this is, that the oncoming wave will pass a slender ship (with a sharp bow) without being disturbed or deformed, whereas it will be diffracted and reflected significantly by a wallshaped barge front. The effect of such diffraction and reflection will be an attenuation of the wave amplitude near the wall.

The significance of this effect is uncertain, but it will be less pronounced with increasing wave lengths.

5.2.4 Hydrostatic Stability

As concluded in /1/ there should be no stability problem for this structure for the conditions investigated. We shall however fully agree with the recommended modeltests to investigate uncertainties related to the operation of the structure. Particularly the influence on stability from waterflow in the CWP and pondwater motion.

5.2.5 Damage Stability

Sufficient damage stability can be obtained by proper arrangements of watertight bulkheads. The proposed arrangement seem to be adequate. However the angle of inclination (heel) of the platform during a possible damaged condition must be less than that giving contact between CWP and the hull side. If this cannot be avoided the CWP element must be strengthen at this position.

5.3 CWP Loads and Responses

5.3.1 Methods of analysis

The general principles of dynamic analysis have been reviewed and can be concurred with.

It is however impossible within the scope of this study to analyse in any detail the special analyses carried out on the CWP. Such a study would represent at least the same amount of work as for the present study of the whole structure.

5.3.2 Lateral Loads

- The drag coefficient $C_D = 0.7 - 0.8$ employed may be on the low side for oscillatory (relative) flow, and also because of the additional non-smoothness of the pipe
- It is unclear how the resulting load from steady current and oscillatory flow is derived. Instantaneous total velocities should be taken as a basis for calculating forces.

5.3.3 Axial loads

The CWP is oscillated longitudinally by the OTEC platform heave motions. The natural period of axial vibration is thus important. Using figures from ref. /1/ (Book III) we obtain for the entire 3000 ft CWP:

$$\text{mass} = 283000 \times 60 \text{ elements} / 32.2 \approx 5.5 \times 10^5 \text{ lb sec}^2/\text{ft}$$

$$\text{stiffness} = 3.3 \times 10^7 / 60 \text{ lb/ft} = 5.5 \times 10^5 \text{ lb/ft}$$

$$\text{Natural period } T_1 = 2\pi \sqrt{\text{mass/stiffness}} \approx 6 \text{ sec}$$

Whether this effect is significant or not, depends upon the magnitude of vertical acceleration (not displacement) of the barge, produced by waves in the 6 seconds period band (for the lowest mode of pipe oscillation).

In particular since the present barge has deep draught, the acceleration response at 6 sec is not necessarily critical. However, only an adequate study of wave-induced acceleration can clarify this point.

Damping properties of the CWP structure will be of great importance in this evaluation. Damping has in ref. /1/ been estimated as 5 % of critical damping ratio. The validity of this estimate should be verified.

A check on oscillations in the water column indicates a natural period of abt. 1.2 sec. Dynamic amplification hereto should thus not be too severe.

HOWEVER, heave - induced pressures and surges may have several effects which are at least to some extent analysed in ref. /1/. The review hereto showed difficulties in finding conclusions, and may thus not be considered complete.

5.3.4 Dynamic stability of the CWP.5.3.4.1 The possibility of joint separation

The joint design allows, physically, that any pipe segment can be lifted a small amount relative to the pipe segment above it. This will happen if the downward acceleration of the latter segment exceeds a certain limit, namely

$$a_{lim.} = g \frac{w_{CWP} - w_{water}}{w_{CWP}}$$

where

g = acceleration of gravity

w_{CWP} = saturated unit weight
of the CWP material
(average)

w_{water} = unit weight of water.

From this formula one can see that $a_{lim.}$ is not necessarily increased by increasing the weight of the CWP. For instance, increasing the weight by increasing the volume of the CWP would have no influence on $a_{lim.}$

As the static weight of the CWP will greatly influence the detail design of the element joints it is necessary to minimize the selfweight of the materials but at the same time the heave acceleration of the barge will require a high unit weight (high $a_{lim.}$). The final solution must therefore be a compromise between the two considerations.

With the proposed light-weight concrete the limit acceleration will be

$$a_{lim.} = g \frac{85 - 64}{85} = 0.247 g$$

$$(= 2,42 \text{ m/sec}^2)$$

The possibility of joint separation depends of course not only on $a_{lim.}$, but also on the real acceleration that may

occur. In this way the arrangement of the pumps may have a considerable effect: If the pumps are fixed to the hull, and the hull is closed above the top of the CWP, the water "plug" inside the pipe may be forced to follow the downward acceleration of the hull. Most likely this would reduce the maximum downward acceleration of the hull, since the submerged weight of the water "plug" is zero. Regarding this problem, see also 5.2.1.2 and 5.3.4.2.

5.3.4.2 The possibility of separation between hull and CWP at the hinge connection.

Principally, this is the same problem as that discussed under 5.3.4. above. However, there is an additional difficulty - connected with the behaviour of the water "plug" inside the pipe: Will it move with the CWP, with the hull, or with the sea water outside the CWP? These three "extreme" situations could possibly occur under the following conditions:

- If the upper end of the CWP is closed, the water "plug" will have to move with the CWP. The resulting unit weight would be close to that of sea water, and the CWP, together with the water "plug", would easily separate from the hull at the hinge connection. With the pumps installed inside the CWP, the situation could approximate to this.
- If the hull is closed above the top of the CWP, the water "plug" will have to move with the hull. The CWP need not necessarily move with the hull, and separation would occur if the downward acceleration exceeds a_{lim} , determined under 5.3.4.1 above.

With the pumps installed in the hull, above the top of the CWP, the situation could approximate to this.

- If both the CWP and the hull are open, the water "plug" need not move with the CWP or the hull. Separation would then occur at the a_{lim} , determined under 5.3.4.1. However,

the acceleration of the hull would be different - under otherwise the same conditions, and therefore the possibility of separation would be different. (Most likely the maximum downward acceleration would be larger, and the possibility of separation larger, than under the second case above).

5.3.4.3 The possibility of unstable oscillations

The question of parametric excitation of oscillations may possibly be of some concern with the CWP. There is a similarity between the CWP and the ordinary pendulum with vertically oscillating support. The latter's movement is governed by the Mathieu equation:

$$\frac{d^2 y}{dt^2} + (a - 2 q \cos t) y = 0$$

which indicates that oscillations with large amplitudes may be possible. See J.P. den Hartog:

"Mechanical Vibrations", page 341. This will be counteracted by damping but it might be worth while to investigate this further, in order to see if the relative rotation at the joints could exceed their capability.

At present we have not found any discussion of this in the APL design /1/. And it is not included in the Paulling computer program as described in OTC-paper 3543 "Frequency-Domain Analysis of OTEC CWP and Platform Dynamics," 1979.

6. ANALYSIS AND DESIGN

6.1 Introduction

This chapter discusses the analysis and detail design of the hull structure, the cold water pipe and the detail connections of the CWP to the hull.

The method, the extent of the analysis and the combination of loadings will be examined together with serviceability and capacity evaluations. And finally the design will be checked according to the given codes, criteria and the present state of the art.

6.2 Hull Structure

6.2.1 Method of Analysis

The analysis and design of the hull structure has been carried out similar to the ARCO - Barge design, which was approved by ABS. The global moments from ABS (ref. chapter 5) has been used to analyse the membrane forces in the different part of the hull structure using a simple beam model. The critical sections have been checked. To analyse the effect of local plate bending from hydrostatic pressure a finite element analysis of half a plate section between transverse bulkhead have been carried out. From this the sections of maximum plate bending can be combined with global ship bending. A beam strip analysis at the elevation of maximum plate bending has been selected to study the effect of hydrostatic pressure, to analyse the effect of variation in internal ballast water, and the effect of different damage conditions.

A transverse beam strip have been used in analysing the base slab, and simple models are used for the check of deck structures.

All such models are very useful and practical methods in the preliminary design, as simple beam analysis very often gives conservative results. However for a final design check the combination of global forces and local plate bending should be checked at all relevant sections to ensure no overstressing. Critical sections may not always occur at sections of maximum plate bending.

The finite element method used for determination of plate bending moments could preferably have been extended to include all forces acting on the section including the global bending and shear forces. Stress combinations could then have resulted from this analyses.

The need for detailed studies regarding stress combination should be evaluated in view of the safety of the design method, but in general the more sophisticated method of analysis used the more optimal design can be obtained.

6.2.2 Load coefficients

As discussed in chapter 3 the load coefficients and combinations used show somewhat conservative values compared with DnV - rules. Although load coefficients are higher and in one case 2,3 times the figures given in the DnV code it seems that the serviceability conditions are governing the design. But this is also the experience from other offshore structures.

6.2.3 Serviceability condition

The condition of providing sufficient prestressing to avoid membrane tension for ordinary loading conditions is very adviseable.

However when checking local stresses from hydrostatic loading the limitation in concrete stresses is not sufficient to limit crack width on tension face. This case becomes critical for a condition where the resulting axial forces becomes zero and the local bending stresses are equal to the allowable tensile stress given in the design specification $f_t = 6 \sqrt{f_c'}$. For such case with only minimum reinforcement, steel stresses will very likely exceed yield. Therefore check on steel stresses or crackwidth control should be included.

For the extreme condition only an ultimate check has been carried out. It is however necessary to include a serviceability control for the 100-years wave to ensure proper function of the structure after this condition. The steel stress limitation as given in DnV-rules /2/ may be a useful recommendation.

It is also important in the serviceability control to check the principal stresses in the vertical walls including membrane shear forces and to ensure that no principal tension will occur in any combination for ordinary loadings.

Fatigue

No fatigue check have been included. Normally no separate fatigue check may be required when the concrete stresses under normal operating condition are always in compression and kept within the limitation of the code.

The particular fatigue problem of offshore concrete structure are related to the possibility for water being pumped in and out from cracks which are opening and closing with the variation in load.

This may therefore be controlled by limiting crackwidth. For the APL hull structure we do not require a separate fatigue check of the concrete if the crackwidth control or steel stress limitation as specified in DnV rules will be included in the design. Otherwise a method as specified in DnV rules may be adopted.

6.2.4 Capacity Check

A capacity check based upon the strain compatibility between concrete and steel using the ultimate strain according to the code specifications is acceptable. The program used for such check should preferably be documented in the calculation.

In the capacity check for local bending (plate bending) two methods are proposed, i.e. checking ultimate capacity from ACI-formulae or using strain compatibility condition. We are much in favour of the strain compatibility method as this gives the best representation of the ultimate conditions. Material factors may be preferable compared to capacity reduction factors.

The ultimate platebending capacity should however represent conservative values as the load coefficients used by APL are greater than those specified in the DnV - code (ref. chapter 3). On the other hand the dynamic contribution of the wall pressure (ref. chapter 5) seems to be underestimated by APL consequently a conservative loadfactor may not give a conservative result. This is particularly the case for upperpart of the walls and for the deck.

6.2.5 Prestressing Scheme

Prestressing will be done by seven wire strand with ultimate strength of 270 ksi (1860 MPa). The loss of prestress is calculated to about 18,5 % which seems to be a reasonable figure. Relaxation of the steel wires which contributes to the greatest portion of the losses should be based upon proper long term testings by the manufacture.

The selected level and layout of prestressing seems reasonable. However, the prestressing program during construction has not been calculated. This should be worked out in view of maximum and minimum strength requirements during this phase.

Placing the prestressing ducts eccentric in the cross-section of the member is a common and useful method to reduce the effect of bending stresses in the section. This has also been proposed by APL for longitudinal cables in the vertical walls to reduce the local bending from hydrostatic loading. But for long cables (370-380 ft.) the effect of friction loss becomes significant due to curvature from variation in the eccentricity. This effect is greatest at midship where also the prestressing requirement is greatest. However, to overcome this problem additional cables can be used at midship position, which is also the suggestion by the designer.

6.2.6 Reinforcement

For a properly prestressed structure the requirement for ordinary reinforcement is often only regarding temperature shrinkage and is given as a minimum reinforcement expressed as a percentage of the concrete cross section. This is also the case for the APL structure, but a percentage ratio of 0,18 of the concrete cross section calculated on average thickness over the whole height of the wall is too

small. We would at least suggest a minimum of 0,15 percent for each side giving a total of 0,3 percent reinforcement.

But the best solution would be to check this according to the recommendations given in DnV-Rules (7.9.2.3) ref /2/ or by ACI-357 (4.5.6) ref. /3/. It should also be noted that the DnV formula includes allowance for waterpressure in cracks. This will, however, become important for the CWP design.

6.2.7 Cover to reinforcement

Concrete cover to the reinforcement is indicated to 2" on external exposure and 1" on internal faces.

The 2" cover on external faces exposed to seawater including ballast is in good agreement with normal practice, 1" cover on internal faces should only be used where no seawater moisture is present, otherwise a minimum figure of 1.5" should be specified. Concrete cover should also be considered in view of the permeability of the concrete and also the size of the aggregate to be used.

Concrete cover to prestressing tendons are normally specified as twice the figures for normal steel (DnV-rules) which gives 3"-4" for structures subject to seawater exposure. ACI-357 specifies minimum figures of 3"-3,5" depending on the degree of exposure. However depending on the quality of the concrete and the ducts to be used less cover may be accepted. For the hull structure minimum cover of 2,25" have been indicated and this means special requirements to the concrete quality i.e. permeability and the quality and type of ducts.

6.2.8 Miscellaneous Effects

- The capacity of ballast compartment walls indicates that limitation of ballast water will be necessary to avoid overstressing of these walls.

Such limitation which must be of a mechanical type represents a danger of failure, which has been experienced on offshore systems from time to time. Unless the system includes some kind of a safety valve we would recommend to design the ballast compartment walls for the maximum physical waterpressure to occur. This will also simplify the mechanical control systems required.

- Detailed design of spherical support of the CWP connection have not been included. For the final design a local finite element analysis would be recommended for this part.
- Support arrangements for the CWP pumps connected to the hull structure should be specified in the final design.

6.3 Design of CWP

6.3.1 Introduction

The cold water pipe (CWP) is an integrated part of the OTEC plant. Important design items are the top hinge, allowing for relative free rotation between the CWP and the platform, the joints between the pipe segments and the pipe segment itself. In the following, possible problems related to structural design and proper functioning of these components are identified and discussed. The evaluation is based on the forces and motions given in "OTEC Pilot Plantship Calculations, Book III. Cold Water Pipe Analysis".

6.3.2 Extent of present design

The presented design is an ultimate strength design, intended to cover two main goals:

- a) To ensure sufficient strength of the CWP at 100-year storm conditions.
- b) To ensure sufficient rotational capacity of the hinges at 100-year storm conditions.

The ultimate design loads are taken in accordance with ACI-318-77, i.e.,

$$U = 1.0 \cdot D + 1.3 E$$

where

U = ultimate design load

D = dead load (permanent load)

E = 100-year environmental load

The presented design approach is considered to be a good basis for estimation of overall dimensions of the CWP.

However, it is obvious that much more refined stress analyses must be performed as a part of the final design in order to verify adequate serviceability and fatigue behaviour. Considering the complexity of the calculations and the rather great consequences of a structural failure, we fully agree with the designer that tests on a large scaled structural model should be carried out. This is of particular interest for verification of adequate behaviour of the hinge between the CWP segments.

6.3.3 Design of CWP segments

The level of vertical prestress has been determined as follows:

$$\phi (P_{\text{eff}} + f_{\text{ct}} \cdot A_c) \geq P_u$$

where

$$\phi = 0.9 = \text{reduction factor}$$

P_{eff} = effective prestressing force

f_{ct} = 250 psi; allowable tensile stress in concrete

A_c = cross section of CWP

P_u = $D + 1.3 E$; ultimate design load, see section 6.2.2.

In the next step, the capacity of the pipe section is checked against combined bending and axial forces according to ACI 318-77. Compliance with ultimate strength criteria is therefore justified.

Both from fatigue and stiffness considerations, it is desirable to limit tension stress and cracking, even at extreme storm conditions. For all load coefficients equal to unity, we estimate the maximum vertical membrane stress to about 950 psi for the typical pipe segment nos. 2 - 19. This value appears to be high. The final design should aim at a membrane stress close to zero.

A few spot checks of initial concrete stresses immediately after anchoring have been carried out. The concrete stresses in a section located just below the spigot,

approximately a distance of 700 mm (2, 3') from the top of segment, is estimated to 2900 psi, or approximately 60 % of the compressive strength. Presently, we have not sufficient information about the lightweight concrete to judge if this value is acceptable.

It is further noted that assumed long term losses in prestress correspond to those estimated for the concrete hull. This assumption may be on the unconservative side as lightweight concrete normally shows higher creep than normal dense concrete. Relevant creep tests should therefore be performed to verify assumptions of effective prestress.

The presented design calculations of the upper connecting details, the lower end of pipe and the bell housing are of rather preliminary nature based on simple equilibrium models. The importance of circumferential prestressing at the bell end in order to control hoop stress and stiffness is underlined. The final determination of stresses and reinforcement in the joint details should be based on finite element calculations, taking proper account of both global and local stresses. Theoretical investigations should be confirmed by model tests, giving failure modes and loads. We do believe, however, that the joint details can be designed with an adequate degree of reliability.

The later submitted documentation received early November explains the detail design of the CWP elements using finite element analysis of the 1/3 model for testing, and indicates great improvements in the actual design of the CWP. This also confirm the estimated behaviour from the previous preliminary design. We will therefore suggest that similar approach should be used for the actual pipe element analysis, including the possible improvements gained from the model test.

Stresses in circumferential direction: In the design of the CWP-segment, stresses in the hoop direction of the pipe must be considered. Such stresses, axial and flexure, may be caused by;

- pressure differences over the wall thickness due to flow of water in the pipe
- pressure fluctuations due to waves
- pressure due to current
- temperature difference between cold water in pipe and outer sea
- axial forces in pipe, causing tensile stresses at bell and compression at spigot end
- bending moments and horizontal forces, causing axial and flexural forces in hoop direction at both ends of CWP segment
- second order moments due to geometrical imperfections

The chosen wall thickness of 4.5" (114 mm), corresponding to $t/R = 0.025$, allows for placing of only one layer of reinforcement in each direction as shown on dwg. CWP-03. This reinforcement, placed in the center of the wall, must provide adequate resistance to bending moment in the hoop direction. Furthermore, adequate distribution of vertical cracks must be assured in order to maintain integrity and durability of the pipe.

In addition hoop stresses may also occur under fabrication, transportation and installation of the segment. We do not feel confident about the hoop strength of the pipe and the crack distributing ability of the hoop reinforcement. This question is accentuated by the high ambient water pressure which might influence the crack distribution see 6.3.4 below, considerably. An analysis of hoop stresses should therefore be carried out even at this stage. The result may be decisive for the design of the pipe.

The finite element analysis carried out for the model test will to some extent include this, but several of the above mentioned effects have not been included in the analysis. We will therefore suggest that all effects causing hoop stresses should be included at an early stage of the design and preferably in the test model analysis.

Another effect to be considered for the final element analysis is that the use of only one layer of reinforcement will introduce considerable stiffness reductions which will not be reflected in a linear elastic model.

It is also recommended to include a stability analysis. The pipe segment may be considered as an orthotropic cylindrical shell with longitudinal stiffening and ring stiffeners at both ends. The lowest ring stiffness is at the spigot end, where the t/R ratio amounts to $8/182.25 = 0.044$.

6.3.4 Minimum reinforcement

The calculation of minimum reinforcement as given in 6.2.6 applies similarly to the CWP as for the hull.

The DnV rules specifies minimum reinforcemens as follows, ref. /2/.

$$A_S = \frac{f_{tm} + W}{f_{sp}} \times b \times d_e$$

where

f_{tm} - mean tensile strength of concrete

f_{sp} - proportional limit of reinforcement

W - waterpressure incracks

d_e - effective tension zone

The effect of waterpressure "W" in cracks as given in the above formula will for the CWP represent a major contribution to the reinforcement ratio. This is particularly the case for elements on greater depth, however the degree of saturation may be allowed for in the determination of "W". Please note that this formula does not indicate that ambient water pressure tend to crack a piece of concrete submerged in deep water, but rather the opposite. Uncracked concrete will appear to have a higher capacity in tension. Cracks introduced for some reason will tend to increase to unacceptable widths in case of loading or restraining. This means that

the ability of the reinforcement to distribute and delimit crackwidths is reduced.

6.3.5 Cover to reinforcements

The minimum cover to reinforcement and prestressing steel will very much depend upon the quality of concrete used and the type of environment. As the concrete for the CWP will be a lightweight type it is reasonable to assume that minimum figures for concrete cover should be at least those specified for ordinary concrete under similar exposure.

The necessity for using greater cover will depend on the results from permeability test and the possibility of using other methods of protecting the reinforcements.

Recommended minimum cover should therefore be, DnV rules ref. /2/.

Ordinary reinforcement	:	50 mm (1.96")
Prestressing ducts	:	the greater of 100 mm (3.94") and the diameter of the duct

This assumes no significant reduction in permeability compared with ordinary concrete or the use of other protective means.

6.3.6 Construction details

In evaluating the final geometry of the hinge details, efforts should be made to simplify details and to avoid abrupt changes in geometry. In this respect we consider the revised design of the bell (sketches dated 9 August, 1979) to be superior to that shown on dwg. CWP-03, dated 31 October, 1978. The revised design allows for simplified arrangement of vertical tendons. Furthermore, all reentrant angles in the bell housing are larger than 90° , giving a more favourable geometry regarding stress concentrations than the older design.

More important details are however the many sharp corners of the element bell and spigot ends. It is recommended that such sharp corners should be avoided as far as practical possible in the final design. As these sharp corners gives a much higher corrosion risk of the reinforcement with the subsequent danger of concrete spalling.

This particular problem is reported by Browne, Domone and Geophegan ref. /13/ and their conclusion support the above recommendation.

Regarding the anchorage of the circumferential prestressing cables we believe that due to the relatively small wall thickness this may call for careful detailing to avoid excessive local stresses. Careful consideration and detail stress control of this anchorage arrangement is therefore recommended.

6.3.7 Typical hinge between segments

The relative rotation between any two mating CWP segments is transmitted as shear deformations in the elastomeric bearings. The joint needs therefore no moveable parts, allowing for a relatively simple design.

First of all the calculations comprise development of basic spring constants to be used in the dynamic analysis of the pipe. The calculations are based on theory of elasticity. Further, the elastomeric pads are assumed to be supported by infinitely stiff structures so that the total deformations must be transmitted by the pads. The above simplifications are of that type usually made in structural calculations, and are not believed to introduce significant errors. However, so far as is partially possible, the planned model tests should comprise measurements of overall forces and displacements in order to support the theoretical calculations.

The formulas and numerical values derived for spring constants have not been checked in detail. We do, however, broadly agree with the basic calculation principles.

Criteria for proper functioning of the elastomeric pads are understood to be:

- a) Any one elastomeric pad should always be in compression
- b) Maximum shear deformation under storm conditions should not exceed 50 - 60 % of the thickness of the pads.

These criteria correspond to accepted practice for design of elastomeric pads.

However larger shear deformations may be acceptable if this can be justified by proper testings.

Based on the above criteria, design curves giving maximum allowable bending moments or horizontal forces in combination with a given axial load have been worked out. Interaction curves for simultaneously acting bending, horizontal and axial forces are not given. We suppose, however, that all effects have been considered when calculating the load effects on a single pad.

We do stress the importance of performing extensive tests of the pad materials in order to establish reliable design parameters. Different from most other structures, easy replacement of a damaged elastomeric pad is not possible. Every effort should therefore be made to obtain suitable materials with sufficient strength, ductility, fatigue and wear resistance.

6.8.8 Top hinge

The design of the top hinge is analogous to a large ball joint intended to provide relatively free rotations between the CWP and the platform hull.

The hinge mechanism consists of three main parts, i.e.

- a) The steel ring, supporting the CWP.
- b) The hemispherical stainless steel shell embedded in the platform
- c) The elastomeric ring, fixed to the steel support ring and sliding on the hemispherical steel structure. In order to reduce sliding friction, the elastomeric ring is coated by teflon on the platform side.

The principle of functioning of this joint is relatively simple. We do not see major problems in designing a joint of required strength. The following discussion is therefore mainly concentrated on questions related to construction, maintenance and possible repair. Furthermore, the possibility of increased friction due to wear of teflon surfaces or fouling of the stainless steel surfaces is addressed.

Some imperfections in geometry must be expected. In other words, the steel ring may not fit the hemispherical shell exactly, resulting in a variable gap to be filled by the elastomeric rings. It is therefore of importance to establish realistic fabrication tolerances, and to provide possibilities for adjusting the position of the elastomeric ring in accordance with obtained geometry of the supporting shell.

Too large built-in imperfections may result in an uneven distribution of support forces and accelerated abrasion of teflon surfaces.

Some difficulties may exist in the fabrication of a continuous elastomeric ring of conical shape. A possible alternative may be to replace the continuous ring by a number of separate bearings evenly distributed. From a construction point of view this alternative is definitely preferable. A segmental solution is also preferable from a maintenance point of view, as replacement of a single bearing is much more easy than replacement of a continuous ring. A possible disadvantage may be that a rubber membrane must be installed in order to prevent leakage of cold water. However, such a membrane may be beneficial also for other purposes. It will give an efficient protection of bearings and sliding surfaces. Experience shows that teflon surfaces are rather vulnerable, and intrusion of debris may cause accelerated wear and damage of the bearings. The designer has indicated the possibility of enclosing the bearing surfaces by a neoprene bag filled with chemically treated water to prevent corrosion and fouling of the bearing surfaces. This neoprene bag will give the necessary tightening of the cold water pool mentioned above. In our opinion a segmental bearing solution combined with a tightening and protecting neoprene bag appears feasible.

Regardless of the solution, the possibilities of increased sliding friction with time must be carefully considered and included in the design of the CWP. Increased friction will give increased restraint of the CWP changing both the dynamic characteristics and the forces in the CWP.

7. CONSTRUCTION AND INSTALLATION

The described construction and installation program represent a comprehensive and thoroughly prepared document. The proposed methods and activities involved are in line with common offshore practice for similar structures.

There are, however, certain remarks and improvements which should be mentioned which will be summarized as follows:

7.1 Construction site

In addition to being a suitable construction site, the deep-water site should preferably have enough water depth for final testing and commissioning of the platform prior to tow-out.

It is of importance that the construction phase includes such testing and commissioning work of the platform prior to tow-out. This testing should include all items necessary when considering the platform as a ship, but also as much as possible of the main power generation plant. Regarding the CWP, we believe that a trial installation of two to three elements should be carried out. This will give a valuable test on the deployment arrangements and the fitting of each element. In addition the reversability of the process can be checked and element joints can be visually inspected afterwards. For such a test water depth of 90 - 100 ft. may be required. If such water depth is not available at sheltered areas, a separate site should be considered. The platform may there either be positioning by tugs or by temporary moorings.

For location of such a site one should not underestimate transportation time for spare equipments and personnel. Also, the time duration of such an operation should not be underestimated, when preparing time schedules.

7.2 Construction Methology of Platform

The construction method of making the bottom structure in drydock and finalizing the concreting in a floating dock is a well known method in offshore construction. It is, however, important to ensure that floatout from drydock can be achieved with reasonable safety. Temporary bouyancy arrangements as aircushions may be used.

The structure when floatout from drydock should be equipped and manned as for a ship. Therefore bilge systems and minimum of safety systems should be installed prior to launching.

For wall construction a stepwise operation has been proposed as an alternative to slipforming. The method implies a number of horizontal construction joints which always represent a weakness in the concrete, although good workmanship and careful preparation and treatments may to some extent improve this. The slipforming, however, is more favourable if dealing with an experienced contractor but the varying wall-thickness with height together with the thin wall at the top makes it more complicated.

The final choice in method should be decided in view of the contractors' experience.

Essential items for the construction at deepwater site (afloat) should be as follows

- hydrostatic stability
- buoyancy and weight
- longitudinal and transversal strength
- mooring forces
- possible ballasting

All these items should be considered at several intervals of this phase. It is recommended to divide the phase into intervals according to the progress of construction.

It is also of great importance to check the structural strength of the platform at different intervals in view of the actual prestressing procedure. Temporary ballasting during the construction phase may very often become critical regarding local strength and therefore need to be properly checked.

Another item for careful consideration is the tolerance requirements regarding the spherical part at the CWP connection. Close control of this will greatly improve the final result.

7.3 Platform deployment

As stated earlier a thorough commissioning should be carried out on all systems possible prior to deployment. Experience has shown that the importance of such commissioning hardly can be exaggerated.

Regarding mooring and towing brackets realistic forces and angles of attack with proper tolerances must be established. Criterias for dimensioning will be wind, current and waves for mooring brackets and normal conditions and survival conditions for towing brackets.

With the above conditions the number and size of tugs will be given. However, narrow towing channels and other restricted waters will also influence the number and type of tugs. The method of disconnecting tugs at final location should result in a quick and nonhazardous operation.

7.4 CWP Construction

Selection of fabrication site will be influenced by several factors as stated by APL, but the importance of a short distance to the platform fabrication site should not be underestimated.

The bell and spigot end of each element represents the critical part regarding strength and is also the most complicated part for casting. Therefore element fabrication should be made such that proper quality and control of the casting of these parts will be obtained. From this, element fabrication in two parts seem favourable. Joining of the two halves will give additional work and introduce a construction joints, but this joint will be located at a favourable position.

The method of joining the two halves together as discussed during meeting on 29 October without reinforcing bars through the joint is not an acceptable method. A proper joint must therefore have a minimum amount of longitudinal reinforcement through the joint to control the crackwidth. This can be done by welding or by use of mechanical couplings of the reinforcement, or alternatively by casting the ring stiffener between the two halves as part of the joining procedure with overlap or welded bars giving two construction joints at each element.

However, fabrication of an element in one part should still be included as a possibility as this may simplify fabrication work. The element thickness and dimensions may also be unfavourable to fabrication in one part. Test program phase II should therefore include a verification of proper casting of thin walled elements of such height.

7.5 Hinge Materials/Assemblies

The major problem in fabrication of the supporting will be the tolerance requirement. It is therefore essential to await the construction of the ring until the platform and the upper CWP element is made, in such a way actual dimension check can be carried out prior to fabrication such to ensure the best fitting ring.

Due to the importance of a proper fitting between the elements a test should be carried out after the casting of the first two elements to check this fitting. Such test may preferably be repeated at a later stage. This test must obviously include the bearing pads and the pressure seal.

7.6 CWP Deployment

Selection of transportation barges for CWP elements should be carried out after proper study of the expected swell conditions on the deployment site.

For towing operations it is recommended that the return period selected for environmental criterias should be taken as three times the expected duration of the operation, with due consideration to reasonable delays and time for return to harbour.

From these conditions the actual barge response and sea-fastening criterias can be derived.

Acceptable weather criteria for deployment of CWP must be established. The response of the barge, the platform and the CWP elements installed must be studied.

Each lifting operation of elements must be considered as a heavy lift and criterias are to be checked prior to each operation. Guiding wires should be considered if necessary. An acceptable position of elements and lifting equipments must be given for standby periods during bad weather.

Cylindrical columns in the CWP deployment frame are proposed guyed during transportation. Rigid stiffeners may be recommended and it may also be necessary to include some stiffeners during deployment of the first few elements and gradually reducing this during the program. For better understanding of this a complete analysis of the response of the platform and the CWP during different phases of deployment program should be carried out.

During the CWP deployment program when the jacks are lifting the entire CWP string, the load transfer should be carried out very slowly and the behaviour of the element joint should preferably be recorded as this also represents the actual load test on each element joint.

7.7 Retrieval

The possibility for retrieval should be included. The risk of overstressing an element is greatest for the upper elements consequently replacement possibilities are therefore more important for these.

The design should, however, not be based upon such a possibility, it should only be considered as an emergency condition.

8. INSPECTION AND MAINTENANCE

8.1 General

The need for proper inspection and control during the design and construction of offshore structures has for long time been considered as absolute necessary to obtain a quality product. To ensure a proper condition for a 20 - 30 years lifetime under offshore exposure similar inspection and control systems becomes necessary during service time.

In-service inspection work must therefore take place under normal operation and often under very unfavourable and difficult conditions. The program must clearly specify the important areas for control, inspection method and equipment to be used and the extent and routines for reportings.

The purpose of the inspection system will be to establish a background for planning the necessary maintenance work which shall ensure continuous operation of the structure and reduce the risk of serious damages.

Planning inspection systems starts at the design stage. Proper recording of areas of high stresses from design and areas of difficult construction or reduced quality due to repairs etc. together with possible experience on similar structures will give the necessary basis for planning inspection work.

The best recipe however is during the design to reduce the areas of high stresses and difficult construction as this reduces the number of critical points, which in turn simplifies inspection and maintenance work.

Experience from the North Sea indicates that offshore concrete structures require a minimum of maintenance work. However inspection work will still be necessary and should aim at covering the whole structure in a period of 4 - 6 years, using annual or biannual surveys. When sufficient evidence regarding the condition of the structure has been obtained, the schedule may be subject to changes. It may however be useful to carry out the inspection on critical parts short after a storm season, hurricane, so that any repair can be done before the next season is expected.

Underwater inspection on offshore structures has normally been carried out as visual inspection by divers, by remote controlled television equipments (RCV) in areas of limited access by divers or by manned submersibles which seems most favourable on greater depth.

For areas in which visual inspection becomes almost impossible, due to access possibilities or because of the continuity in the operation, instrumentation systems have been installed to measure the actual behaviour. Such systems has however shown varying degree of success.

For concrete structures visual inspection should in general focusing on the following items:

- surface cracking, larger than 0,3 - 0,5 mm
- spalling of concrete
- deterioration of concrete
- corrosion of reinforcement
- corrosion of pipes penetrating the concrete faces or steel items partly embedded in concrete
- condition of possible coatings

It is further important that the structure in question will be properly marked with signs for easy orientation and references during underwater inspection. This reference system may preferably be the same as used in the design and construction and need only to be marked sufficiently for underwater use.

Further information on inspection of offshore concrete structure is given in Appendix C.

8.2 Hull Structure

The general principle of the inspection and maintenance system have been outlined above. The maximum draft of the hull structure is 65 ft. Underwater inspection can therefore most efficiently be carried out by divers operating from a separate diving vessel or from the hull structure, for the external faces. For internal inspection manholes, access tunnels etc. should be arranged in order to simplify access to inner concrete surfaces.

The purpose of the inspection of the hull structure should be to establish a general condition of the concrete with due account to the items mentioned above. But important are also items which contribute to water tightness and stability of the structure.

8.3 Cold Water Pipe

The CWP is completely submerged and extends down to depth of 3000 - 4000 feet. The inspection system may therefore preferably be divided into areas according to the method of inspection. The upper part of the CWP seems most suitable for divers while submersibles seem at present the only solution for the greater depth.

For internal inspection of CWP during operation (flow of 6 feet/sec.) RCV seems the most realistic method. However the necessary support arrangement for RCV may complicate this such that with present experience the CWP may only be inspected during shut-down periods.

The areas of inspection in addition to the general conditions as given above will be the element joints. The condition of the neoprene bearings, the sealing ring and the concrete structure of the pipe at this level will be subject for close inspection.

Similarly the gimble bearing of the ball joint connection between the CWP and the hull structure will be subject for close inspection. The neoprene ring bearing with the teflon sliding face represents one of the critical parts of the gimble bearing. If this bearing and teflon surface can not be constructed of replaceable elements, a monitoring device and an inspection system must be established to give regular information about its condition. We believe that due to the difficulty of visual inspection the monitoring system must give the required information about the condition of teflon cover. For instance continuous thickness measuring may be used.

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APPENDIX A

APPENDIX A

Documentation submitted by APL

- A Baseline Design of an OTEC Pilot Plant Ship
APL/JHU SR-78-3 May 79
- OTEC Pilot Plant Ship Calculations, Book I, Hull
Seakeeping, Stability and Hydrodynamics
- OTEC Pilot Plant Ship Calculations, Book II Hull Structural
Design
- OTEC Pilot Plant Ship Calculations, Book III Cold Water
Pipe Analysis
- Preliminar Engineering Design of OTEC
Vol. A: Executive Summary and Detailed Report APL/JHU
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- Preliminar Engineering Design of OTEC
Vol. C: Specifications, APL/JHU SR-78-3C Nov. 78
- Preliminar Engineering Design of OTEC
Vol. D.: APL/JHU SR-78-3D. Nov. 1978
- Report on Preliminary Engineering Design.
APL/JHU, 20th November, 1978
- Lightweight Concrete Development Program, Phase I,
APL/JHU SR-79-1, April, 79
- Development of a Lightweight Concrete for OTEC Coldwater Pipes:
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- Meeting with APL/ABAM at V.S.E. Corporation in Alexandria, Virginia 29 October, 1979.
- OTEC Environmental Package, Semi-Annual Updated Report. September 1979, received in meeting 29.10.1979.
- Description of finite element analysis for CWP model test. Letter from ABAM of 6 November, 1979.

APPENDIX B

EXTRACT FROM DNV "Ships' Load & Strength
Manual, Section 4, Ship Motion and Local
Loads" pages 27-34

SECTION 4 SHIP MOTIONS AND LOCAL LOADS

A. General	D. Liquid Pressure in Full Tanks
A 100 Introduction	D 100 General
A 200 Definitions	D 200 Static pressure
A 300 Ship motions	D 300 Dynamic pressure
B. Sea and Weather Pressure	E. Liquid Pressure in Partly Filled Tanks
B 100 General	E 100 General
B 200 Static pressure	E 200 Definitions
B 300 Dynamic pressure and forces	E 300 Swashing pressure
B 400 Slamming pressure in bottom forward	E 400 Liquid impact pressure
B 500 Bow impact pressure	E 500 Pressure in tanks with restriction to the filling height
C. Pressure and Forces from Bulk Cargo, General Cargo, Stores and Concentrated Loads	E 600 Effective generating length (tank fetch) l_t and b_t
C 100 General	F. Pressure on Watertight Subdivision Members
C 200 Static pressure and forces	F 100 Definitions
C 300 Dynamic pressure and forces	F 200 Flooding pressure

A. General

A 100 Introduction

101 In this section it is given general expressions for calculation of:

- Ship motions.
- Static and dynamic pressures and forces.
- Extreme static pressures.

102 The expressions for ship motions and dynamic pressures and forces have been based on general ship theory, and the formulae given are considered to represent values corresponding to a probability level of $Q = 10^{-8}$ for wave spectra in the North Atlantic.

103 The impact pressure $p_{s\ell}$ is given for:

- Slamming in bottom forward.
- Bow impact.
- Impact pressure in partly filled tanks.

The expressions for $p_{s\ell}$ are values for the "tail pressure" (see Sec. 3) based on experimental and theoretical investigations calibrated against damage experience from ships in service, applying the impact strength approach given in Sec. 5 D.

A 200 Definitions

201 The point where the pressure is to be calculated is defined for various strength members as follows:

For plates:

- Midpoint of horizontally stiffened plate.

- Half the stiffener spacing above the lower edge of the plate or plate support, for vertically stiffened plates.

For stiffeners and girders:

- Midpoint of span, when equally spaced.
- Centroid of supported area, when unequally spaced.
- In case of direct stress analysis of vertical members, the pressure distribution is considered.

For pillars and cross ties:

- Centroid of supported area.

202 Symbols

General symbols are defined in Sec. 1. Other common symbols applied in Sec. 4 are as follows:

p_s	static pressure in seagoing condition (N/mm^2).
p_h	extreme static pressure (N/mm^2)
p_d	dynamic pressure (N/mm^2)
p_f	flooding pressure on watertight compartments (N/mm^2)
p_{sl}	impact pressure (N/mm^2)
F_s	static force (N)
F_d	dynamic force (N)
q_D	mass of distributed load in (tonnes/m^2) on inner bottom and deck.
ρ	density of cargo in (Mg/m^3), (t/m^3) for sea water $\rho = 1,025$ (Mg/m^3), (t/m^3)
g	acceleration of gravity $9,81$ (m/sec^2)
δ	angle of repose of bulk cargo (degrees)
x	transverse distance from centreline of ship to point considered (m).
y	vertical distance from baseline of ship to point considered (m).
z	longitudinal distance from AP, FP or $0,05L$ aft of amidship to the point considered (m).
h	vertical distance from top of cargo to point considered (m)
l	longitudinal distance from end of tank to point considered (m).
b	transverse distance from side of tank to point considered (m)
h_λ	effective wave height for the ship considered (m).

For $L < 350$ m h_λ is taken as the smaller of:

$$\begin{aligned}
 h_\lambda &= D \\
 &= 13 - \left(\frac{250 - L}{105} \right)^3 \\
 &= 13 - \left(\frac{L - 250}{105} \right)^3
 \end{aligned}$$

For $L > 350$ m is taken:

$$h_{\lambda} = \frac{227}{\sqrt{L}}$$

C_v = speed reduction coefficient for ship in heavy sea

$$\approx 1 - \left(\frac{750}{1000 + L} \right) \frac{h_{\lambda}}{D}$$

- a acceleration as fraction of g
- a_v ship's vertical acceleration (see A 307)
- a_t ship's transverse acceleration (see A 308)
- a_g ship's longitudinal acceleration (see A 309)
- V ship's service speed (knots)

A 300 Ship motions

301 The ship motions expressed as angles, periods and accelerations are given in the following for an estimated probability level of $Q = 10^{-8}$ in the North Atlantic taking into consideration speed reduction and proper handling of the ship in heavy weather.

302 Heave.

Heave acceleration a_h :

$$a_h = \frac{2 \left(\frac{V C_v}{\sqrt{L}} + 1,2 \right)}{C_B} \frac{h_{\lambda}}{L}$$

303 Pitch.

Pitch period T_p :

$$T_p = \frac{0,81 \sqrt{L}}{\sqrt{\frac{V C_v}{\sqrt{L}} + 1,2}} \quad (\text{sec.})$$

Pitch angle Θ_p :

$$\Theta_p = \frac{80 h_{\lambda}}{C_B L} \quad (\text{degrees})$$

Pitch acceleration a_p :

$$a_p = \frac{8,5 \left(\frac{V C_v}{\sqrt{L}} + 1,2 \right)}{C_B} \frac{h_{\lambda}}{L} \frac{R_p}{L}$$

where:

R_p = radius from waterline 0,05 L aft of amidship to the point considered (m).

304 Surge.

Surge acceleration a_s :

$$a_s = 0,15 \frac{h\lambda}{D} \quad \text{for forward direction forces}$$

$$= 0,1 \frac{h\lambda}{D} \quad \text{for aft direction forces}$$

305 Rolling.

Rolling period T_r :

$$T_r = \frac{2 k_r}{\sqrt{GM}} \quad (\text{sec.})$$

$$\approx 3 \sqrt[3]{B} \quad \text{for ship in ballast condition}$$

Rolling angle Θ_r :

$$\Theta_r = 20 \sqrt[4]{\frac{30}{B}} \quad (\text{degrees})$$

Rolling acceleration a_r :

$$a_r = \left(\frac{2\pi}{T_r} \right)^2 \frac{\pi \Theta_r}{180} \frac{R_r}{g}$$

Heeling angle Θ :

The maximum heeling angle may be expressed as:

$$\Theta = 30 \sqrt[4]{\frac{30}{B}} \quad (\text{degrees})$$

where:

GM metacentric height (m)

 k_r transverse radius of gyration including entrained water (m). $\approx 0,39 B$ for ships with even transverse distribution of mass $\approx 0,35 B$ for tankers in ballast $\approx 0,25 B$ for ships loaded with ore between longitudinal bulkheads R_r radius from axis of roll to the point considered (m)

The axis of roll may be taken:

- At the centre of gravity C.G. of the ship, when C.G. is lower than the waterline.
- Midway between C.G. and waterline, when the waterline is lower than C.G.

$D/2$ will normally be a practical mean value for loaded and ballast condition of the ship.

306 Sway and yaw.

These accelerations will be small and need normally not be considered in the strength calculations.

307 The ship's vertical acceleration a_v .

The vertical acceleration resulting from pitch and heave at a phase angle of 60° may be approximately expressed as:

$$a_v = \frac{\left(\frac{V \cdot C_v}{\sqrt{L}} + 1,2 \right)}{C_B} \cdot \frac{h_\lambda}{L} \left(8,5 \frac{z}{L} + 1,4 \right)$$

where:

z = longitudinal distance from $0,05 L$ aft of amidship to point considered (m).

$\left(\frac{z}{L} \right)_{\min} = 0,25$ should normally be used within the amidship region.

308 The ship's transverse acceleration a_t .

The transverse acceleration at a rolling angle Θ_r may be expressed as:

$$a_t = \left(\frac{2\pi}{T_r} \right)^2 \cdot \frac{\pi \Theta_r}{180} \cdot \frac{(y - y_r)}{g}$$

where:

$$\Theta_r = 20 \sqrt{\frac{30}{B}} \quad (\text{degrees})$$

y_r = vertical distance from baseline to the axis of roll (m)

T_r = roll period (see 305).

309 The ship's longitudinal acceleration a_ℓ .

The longitudinal acceleration resulting from surge and pitch in phase may be expressed as:

$$a_\ell = \frac{h_\lambda}{L} \left[\frac{8,5 \left(\frac{V \cdot C_v}{\sqrt{L}} + 1,2 \right)}{C_B} \cdot \frac{(y - d)}{L} + \frac{\psi L}{D} \right]$$

a_ℓ = 0,25 for the case of collision or grounding

where:

ψ = 0,15 for forward direction forces
0,10 for aft direction forces

d = draught (m) for the case considered

B. Sea and Weather Pressure

B 100 General

101 The following pressures are considered:

- Static and dynamic sea pressure p_s and p_d and wind forces (B 200 and B 300).
- Slamming impact pressure in bottom forward $p_{s,l}$ (B 400).
- Bow impact pressure $p_{s,l}$ (B 500).

102 The sea and weather pressures are given in the following irrespective of any counterpressure from inside cargo.

B 200 Static pressure

201 The static pressure from the sea below waterline is:

$$p_s = \rho_{sea} \cdot g \cdot \frac{(d - y)}{1000} = \frac{(d - y)}{100} \quad (\text{N/mm}^2)$$

202 The static pressure from snow and ice on horizontal surfaces may normally be taken as:

$$p_s = 0,25/100 \text{ (N/mm}^2\text{)}$$

B 300 Dynamic pressure and forces

301 The dynamic pressure caused by the relative movement between ship and sea are given for:

Ship in upright condition (see 302)

Ship in heeling condition (see 303)

For strength calculation of local structural members the greatest pressure of these two conditions is to be considered. For larger integrated girder systems the dynamic pressure for the upright condition may normally be used.

302 Ship in upright condition:

where:

$$p_{d(wL)} = \frac{h_\lambda}{100} \left[0,8 + \frac{2,5}{C_B} \left(0,8 + \frac{V}{6,1 \sqrt{L}} \right) \left(\frac{z}{L} - 0,25 \right) \right] \quad (N/mm^2)$$

z longitudinal distance from 0,05 L aft of amidship to point considered (m).

$$\left(\frac{z}{L} \right)_{min} = 0,25$$

C 1 for ship's sides
0,8 for horizontal decks

303 Ship in heeling condition
(normally of interest for larger ships):

Above loaded waterline

$$p_d = \frac{x \sin \Theta - (y - d)}{100} \quad (N/mm^2)$$

Below loaded waterline

$$p_d = \frac{x \sin \Theta}{100}$$

where:

$$\Theta = 30^\circ \sqrt{\frac{30}{B}} \quad (\text{degrees})$$

304 The dynamic wind force applicable for deckhouse and other erections on the ship deck.
may be expressed as:

$$F_d = C_s g \frac{v^2}{16} A \quad (N)$$

where:

A projected area (m^2)
 v wind velocity ≈ 45 (m/sec.)
 C_s shape coefficient
1,0 for cylindrical structures (masts etc.)

$$= \left[2 - 0,5 \left(\frac{l_v}{2b_v} \right)^3 \right] \left(0,5 + 0,1 \frac{h_v}{b_v} \right)$$

for deck and erections
with sharp corners, with
the following limitations:

$$\frac{l_v}{b_v} \max = 2$$

$$\frac{h_v}{b_v} \max = 5$$

l_v , h_v and b_v are the length, height and breadth of the structure respectively, in the considered wind direction (see Fig.).

B 400 Slamming pressure in bottom forward

401 The slamming pressure on bottom panel forward is given as:

$$p_{sl} = \frac{c_1 c_2}{200} \frac{B_b}{d_B} \left(0,4 - \frac{L}{2500} - \frac{z}{L} \right) \text{ (N/mm}^2\text{)}$$

$$c_1 = \sqrt[3]{L} \text{ for } L \leq 150 \text{ m}$$

$$c_1 = \sqrt[3]{255 - \frac{L}{2}} \text{ for } L > 150 \text{ m}$$

$$c_2 = \sqrt{\left(\frac{L}{d_B} \right)^3} \text{ maximum 250}$$

d_B design ballast draught (m) at 0,1L abaft F.P.

B_b the breadth of the bottom (m) at the height 0,15 d_B above the baseline measured at the cross section considered.
maximum = 1,35 d_B or 0,55 $\sqrt{L}$ whichever is smaller.

z distance (m) from F.P. to cross section considered.

The assumed variation in slamming pressure is shown in Fig.

APPENDIX C

VERITAS CONFERENCE

**27TH—28TH NOVEMBER 1979
HØVIK, OSLO**

**«OFFSHORE INSERVICE
INSPECTION AND MONITORING
— A Basis for Efficient
Operation and Maintenance»**

Concrete Structures

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CONTENTS:

1. POSSIBLE DETERIORATION OF CONCRETE
2. INSPECTION METHODS
3. NDT - TECHNIQUES
4. FOUNDATIONS
5. EXPERIENCES FROM PRACTICE
6. THE BRENT B INSTRUMENTATION PROJECT
7. RECENT RESEARCH ON REBAR CORROSION
8. CONCLUDING REMARKS

1. POSSIBLE DETERIORATION OF CONCRETE STRUCTURES

Considering the costs inherent in construction and inspection onshore compared to underwater inspection and maintenance, the philosophy is clear:

The structure should be built and inspected onshore to such a standard that no maintenance is likely to be necessary after immersion.

For concrete structures in the North Sea this philosophy has been implemented with great success. For this reason knowledge about their possible deterioration has to be based on experiences with other coastal structures of inferior material quality or design and from theoretical textbooks.

The mechanisms of the degradation processes involved can be summarized as:

On concrete

- a) Chemical attack by the sulphates in the sea water on the calcium hydroxide ($\text{Ca}(\text{OH})_2$) or tricalcium aluminate hydrate (C_3A) components of the hardened cement matrix. This can result in disruption or softening of the concrete, but in general the problem is less severe in marine conditions than in sulphate bearing groundwaters. In submerged concrete there will also be some leaching of the more soluble substance from the cement matrix, mainly the calcium hydroxide. This by itself need not necessarily lead to a loss of concrete strength, but may result in reduced alkalinity and possible steel corrosion.
- b) Freeze-thaw damage on exposed saturated concrete with high porosity, which weakens and may disrupt the cement matrix.
- c) Fatigue mainly associated with water pumping in and out of possible cracks in areas of pronounced repetitive tension forces

- d) Abrasion caused by wear of solid particles carried by the flowing water or by cavitation

On reinforcing steel

- a) The chlorides in the sea water will penetrate the concrete cover and may then destroy the steel passivity resulting from the highly alkaline concrete environment. With sufficient oxygen present, the steel will then corrode, an expansive reaction, which may then generate sufficient tensile stresses to crack and spall the concrete cover. As further discussed below, the likely-hood of maintained corrosion of reinforcement in submerged concrete is questioned by recent research.
- b) Finally the structure might fail due to overload caused by design errors. misuse or accidents.

Apart from the structural concrete the following attachments might - and are in fact more likely to - be deteriorated

Foundation scouring

If the structure is founded on fine sand and platform or adjacent platforms have unfavourable configuration, scouring and undermining of the base slab might occur.

Fixing plates and steel inserts

The reinforced concrete will have a higher electrochemical potential than naked steel. Thus the naked steel tend to act as an anode and thus corrode seriously unless appropriately protected.

Coatings

In practice coatings tend to be less durable than the concrete which it is intended to protect.

It is obvious that the latter three items should be considered significant items and subject to inspection. Sound structural concrete as used in North Sea structure seems not valuerable to deterioration and the significant areas should be limited to areas where inferior quality has been revealed during the design and construction phase.

In case of damage or unsuccessful construction which has to be repaired in the construction phase, the repair should always be considered a weak point. Furthermore the splash zone, construction joints, highly stressed zones and the shaft deck connection should be considered in the definition of significant items.

2. INSPECTION METHODS

The size of concrete gravity platforms immediately imposes practical restrictions on selection of areas and elements to be inspected on the structure. As an example the Condeep Statfjord A-platform has an external surface area of the concrete substructure of approx. 50.000 m². The geometry and size of the caisson-tower type structure will normally give rise to problems with position identification on the structure under water for divers, submarines or remote controlled video equipment. Therefore it is strongly recommended that a grid system on the concrete surface should be established by means of markers easily recognized by the above means of inspection. The size of such a grid is to be designed with view to visability and according to the need for detailed inspection, but it should not exceed 10 by 20 meters horizontally and vertically. The grid of markers should preferably be established in the design/construction phase of the structure.

With view to availability of access and also potential danger for long term material deterioration of a concrete structure of the oil/gas production platform type, in-service inspection of the external surface of the structure is normally considered to be sufficient. However in the tower-caisson type of structure, inspection internally of base of shafts may be required depending on the SCR data.

With regard to in-service inspection of concrete structures, we are presently of the opinion that sound practical principles for inspection should be preferred before advanced NDT-techniques or built-in instrumentation proposed by several recommendations. For the sake of effective inspection operations, we strongly emphasize the importance of personnel involved in such inspections to be familiar with and well prepared for the objects to be inspected. In general DnV-surveyors are engineers with additional training as divers and NDT-operators. They are also experienced in inspection of similar structures during construction. In the course of inspection they shall hence be in the position to perform spot-checks by diving when deemed necessary.

Inspection may be carried out by the following means:

ATMOSPHERIC ZONE:

- visual inspection from removeable working platforms, cages, boats etc.

SUBMERGED ZONE:

- visual inspection by means of
 - i) remote controlled television equipment (RCT)
 - ii) divers
 - iii) submarine
- instrumentation

Removal of marine growth in the submerged zone is normally to be allowed for on items/locations to be inspected.

In general primary inspection by RCV-equipment is preferred. Inspection by divers or by means of submarine should be supplementary to RCV and carried out where deemed necessary from immediate evaluation offshore of RCV-inspection. Simple mechanical tools e.g. hammers, scrapers and measuring tape should be standard equipment for divers. Pneumatic chiselling equipment should be available for further investigations of suspicious findings from visual inspection.

Selection of methods and equipment should give consideration to the following factors:

1. Properties, location and accessibility of inspection items and potential defects that may be found.
2. Type of inspection, i.e. I, II, III required
3. Availability of reliable equipment

For concrete structures inspection types I and II only will normally be sufficient. Great emphasis is to be put on the quality of visual inspection with qualified personnel, especially with respect to common terminology, photo documentation and sampling. To establish a common terminology for description of inspection findings Ref. 5 is recommended.

From a practical and a cost point of view the visual inspection supplemented by a simple scrape etc. is the more feasible method for under water inspection. It seems fair to assume that all major sources of damage to concrete structures will be visible before they reach an extent jeopardizing the integrity of the structure.

- Overloading result in excessive cracking
- Rebar corrosion, surface staining, cracking and spalling
- Deterioration, - loose concrete valuerable to scaping etc.
- Sulphate attack, - crumbling of the surface

In cases of dubious appearance cores drilled and closely examined in laboratory seems to be a more reliable basis for evaluation of the platform condition than semi - developed NDT equipment applied under water, i.e. under the most difficult working conditions credible.

3. NDT TECHNIQUES

The following types of NDT equipment have been accentuated for possible use in the North Sea. All the techniques mentioned have demonstrated their feasibility in dry conditions, but the experiences for under water applications are limited and occationally somewhat dubious.

Ultrasonic testing

The concrete quality is estimated on the basis of sound velocity measurements. The distance could be through the thickness of the concrete member or between two points on the same surface. Above water the method is also claimed to be feasible for crack detection, lamination etc. Probably its potential for lamination detection is the main possible merit of the method. Under water the method has so far not given acceptable results. The method is also time consuming and the present stage feasible only in special cases.

Gamma ray backscatter

The reflected intensity of gamma radiation can be related to the concrete density. The density in itself is not a feasible parameter to evaluate the strength of sound concrete, but changes in density may indicate deterioration of the concrete.

Even if the method is simple it is time consuming and its application should be restricted to areas which are identified to call for special investigations. It is questioned whether this method should be preferred to drilling of cores which can be examined in laboratory onshore.

Radiographic survey

In cases of access to both sides of the concrete surface radiographic surveys may be a feasible tool to reveal voids in the concrete, inadequately grouted cable ducts etc. Basically such shortcomings should have been observed prior to immersion of the structure. Thus this should only be used under water in special cases.

Rebound hammer

This tool is based on measurements of the rebound of a body thrown on the surface. This can be related to the concrete elasticity which again is related to the concrete strength.

Even if efforts have been made to modify conventional hammers for under water applications, the scattering is large, and the main application of the method should be to compare readings from one survey to the other. Other disadvantages are

- time consumption
- divers have to take the readings himself
- results concern mainly the surfaces; the properties of the inner parts of the concrete is less influenced
- a smooth clean surface is required, this is normally not the case for slipformed surfaces after a few years in sea water

Lok-test devices

The Lok-test principle is based on the force necessary to extract a cast in circular steel plate from the concrete by utilizing a jacking system which exerts a force perpendicular to the surface of the concrete which is related to its strength. Broadly speaking the disadvantages of the Lok-test methods are the same as for the rebound hammer. Furthermore it requires a lot of devices to be cast into the concrete in the areas anticipated to call for in-service control. Once overloaded a specimen is invalid for further tests. On the other hand the accuracy of the Lok-test method is higher than for the rebound hammer.

Electrochemical potential measurement

Feasible electrodes are attached to the concrete surface and the reinforcement to identify electrochemical potentials which may be related to the reinforcement corrosion. Access to the reinforcement might be a problem for the practical application of the method. The method has not been demonstrated feasible for under water applications.

By the application of preinstalled reference cell and counter electrode cast into the structure during construction the potential and polarisation of the steel can be monitored. This system also permits macrocell activities to be revealed. However, at present the investments involved make this equipment unfeasible for anything but research purpose. From a practical point of view equipment to measure differences in electric current density across the concrete surface to reveal areas of rebar corrosion seems more promising. However, uneven distribution of reinforcement and other embedded steel presence of anodes and other current sources etc. might render the results questionable without further verification of the method.

Concrete cover

Cover to reinforcement should have been checked prior to immersion and under water inspection is necessary only in exceptional cases. If necessary it can be measured by a magnetic reluctance meter which seems to be a simple and quick procedure.

Built-in instrumentation

Various forms of built-in structural instrumentation have been proposed in order to monitor in-service performance. This could be built-in sensors for ultrasonic pulse measurements, strain gauges etc. Though not disregarding their importance in special cases, we are presently of the opinion that, with view to long term reliability and processing of data, such instrumentation does not represent practical substitutions for visual inspection as outlined above. However, minimum long term instrumentation normally required for each installation as mentioned briefly below will serve as valuable supplementary information for the foundation of structure.

The stress level of certain platforms has been controlled by preinstalled strain gages, pressure cells etc. These devices seem well suited for research purposes, but hardly as tools for routine platform monitoring.

4. FOUNDATIONS

For the foundation soil, visual inspection or surface measurements give very limited information (except on scour). Instrumentation is therefore necessary in order to check the performance of the foundation. Such measurements will be of particular value in evaluation of the performance of the foundation during extreme loadings and can be used to predict the performance of the foundation under even more severe conditions.

For typical gravity structures that are presently under certification by Det norske Veritas the following geotechnical instrumentation is considered justified.

Long term settlements

1. One settlement gauge is desirable. The method of observation should allow continuous settlement observations from the time of installation of the structure and for several years.

Motions at base and deck level

2. In order to study soil/structure interaction angular and linear accelerometers or inclinometers are desirable. The instruments must be capable of recording dynamic movements due to wave action. To define the motions of the platform, a total of five gauges may be required at the base level

(three linear and two angular) and three gauges at the deck level (two linear and one angular) giving a total of eight gauges.

Pore water pressure in soil

3. Depending on soil conditions the pore water pressure should be measured at five to ten locations in the soil mass under the structure.

Scour monitoring

Scouring is difficult to predict on the basis of theoretical considerations. The cost of installation of a scour protection which eliminate any theoretical risk will be high. But as development of scouring jeopardizing the foundation stability requires considerable time, normal practice has been to postpone scour protection installation pending scour observations in practice. These observations are carried out by visual inspections from submarine. As reference is used vertical sticks penetrated into the sea floor in appropriate distances from the platform.

So far only one platform has called for scour protection installation.

5. EXPERIENCES FROM PRACTICE

The Ekofisk tank was installed in 1973 and is followed by 12 more North Sea concrete structures. All these structures have been subject to formal or informal inspections. Some inspections have been based on rational programs as described above, others have seemingly been based on a more arbitrary selection of items inspected.

Some inspections have been restricted to the atmospheric zone. Under water inspections have mainly been carried out by RCV and by submarine. When possible damage has been investigated, diver inspection has been carried out.

The findings of the inspections are in no way exciting. In fact the findings are very similar for all the platforms:

- Inspection of caisson roof and inside of drilling shafts has been hampered by debris and drilling mud
- The splash zone and upper part of the shafts have been covered by soft marine growth.
- Embedded steel plates inserts and appurtenances are corroded, but seemingly not seriously impaired
- Steel appurtenances are in certain cases incomplete
- The concrete surfaces showed on excellent condition without any harmful cracks or defects
- Possible cracks has a tendency to heal by sedimentation of magnesium and chalk compounds

One or two minor scratches have been observed on several platforms, mainly caused by external impact.

Such cases might be:

- In one case a 36" drain caisson was dropped on the concrete cell roof and locally spalled off the concrete to a maximum depth of abt. 10 cm.
- A riser improperly fixed rubbed the shaft wall and caused a minor abrasion

- On several occasions accidental ship impacts have resulted in score marks of a few cm depth.
- In one case the formwork, which was made of plain concrete and which was not removed, eventually spalled off after a few years of splash zone exposure. As this shuttering had no structural significance this spalling did not reduce the quality of the structure
- In one case 1-2 cm of concrete spalled off during a fire inside one of the shafts.

The majority of these cases have been repaired, even if a profound study of their possible consequences might have concluded them to be completely harmless. In general the influence of minor defects to be overestimated by personnel not skilled in concrete structural design. In certain cases large volumes of reports and investigations have been prepared to study defects obviously lacking any practical significance.

6. THE BRENT B INSTRUMENTATION PROJECT

To obtain a mature understanding of the performance of offshore concrete structures a great full scale measurement project has been carried out monitoring its behaviour in service.

The principal purpose of the project was to collect data from the Brent B Condeep platform which could be used

- to verify the soundness of the design principles currently in use for concrete gravity structures in the North Sea.
- to improve the quality and accuracy of the design methods used for such structures
- to predict possible damages of this and other platforms of similar design

The design of the shafts on the platform was based on considerations of the ultimate strength, fatigue strength and serviceability requirements. As it turned out, the serviceability requirements had the most significant influence on the design.

The bending moments at the foot of the shafts were estimated for a 100 year design condition, based on the moments calculated from observed strains and on extrapolation of the wave condition. Bearing in mind the uncertainties in this procedure, it was concluded that the measurements show that in a design sea state the shafts are expected to experience load effects of the same or slightly smaller magnitude than assumed in the design.

The design capacity, however, is more than twice the design load effect. Hence, the safety of this concrete section is considerably higher than required by the design code.

The measurements indicated that the shafts will experience only compressive stresses, except for possible small edge stresses in tension due to local bending at the base of the shafts. The cyclic stress variation in the reinforcement will therefore, be limited and well below the fatigue strength endurance limit.

A general conclusion of the project is that there is no indication in the measurements or the analyses that the platform will perform less satisfactorily than anticipated at the design stage. This conclusion is valid for the foundation soil, as well as for the structure itself.

7. RESENT RESEARCH ON REBAR CORROSION

A considerable part of the reinforcement in a concrete platform is introduced to limit the surface crack widths which used to be considered the major cause of rebar corrosion. As the traditional design criteria appeared to be established on basis of intuition rather than strict research, considerable efforts has been carried out in Germany, in UK and in Norway to obtain a more mature understanding of the correspondance between crack widths and rebar corrosion.

The preliminary conclusions of these studies related to fully submerged concrete of high and even quality may be summarized as follows:

- Rebars corrosion tends to be initiated at an earlier time close to the wider cracks than otherwise
- The rate of corrosion after initiation depends mainly on the concrete quality and is independent of the crack width
- Corrosion products, i.e. Mg and Ca compounds from the concrete deposited in the crack, hamper the maintained corrosion. The corrosion rate is decreasing and after 1-2 years the corrosion is almost stopped
- The final amount of corrosion is broadly independent of the crack width at least if the load is static and crack widths less than 0,5 - 1,0 mm

These findings are supported by independent theoretical and experimental research in all the three countries mentioned above.

DnV has contributed to this research by potential measurements of precracked reinforced concrete specimens.

Also DnV's observations of the anode consumption of the cathodic protection of the concrete platforms support these conclusions. As long as the naked steel in the cracks is in an active phase they will be protected by the anodes. This will be recorded as increased current drain. After 1-2 years, consumption of anodes has been observed to stabilize at a lower value. This may be considered an indication that the process observed on small test specimen also take place in large real life structures.

8. CONCLUDING REMARKS

In summary it may be stated that the principle is to have the concrete structures controlled to meet the design criteria prior to the immersion in the North Sea. The design criteria are chosen to achieve a structure which can stand its intended life without any kind of deterioration, maintenance or repair.

Recent research has confirmed the design assumptions as regards load effects in the structure. There is also strong indications that the design to ensure the durability e.g. related to rebar corrosion is a conservative one. Finally it may be concluded that all main elements of the structure are either preloaded during temporary phases during construction and installation to load levels higher than they will ever experience during operation or their design to meet the serviceability criteria has resulted in considerable strength reserves.

For this reason it is not unlikely that in future the amount of in service inspection of concrete structures may be reduced, that maintenance will be found unnecessary and that repairs will be restricted to accidental damage associated with ship impacts etc.

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