OCEAN THERMAL ENERGY CONVERSION: COLD WATER PIPE
PRELIMINARY DESIGN PROJECT

Task 2. Analysis for Concept Selection

MASTER

April 1979

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OCEAN THERMAL ENERGY CONVERSION
COLD WATER PIPE PRELIMINARY DESIGN PROJECT

TASK 2. ANALYSIS FOR CONCEPT SELECTION

1979 APRIL

Prepared for
NOAA/DOE

Prepared By

TRW
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Redondo Beach, Calif.

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1. INTRODUCTION AND SUMMARY

The Ocean Thermal Energy Conversion (OTEC) Program plans to deploy seabased test facilities and demonstration plants, ranging in size from 1 MWe to 400 MWe, during the next several years. An important part of these facilities will be the subsystem for delivery of the cold water to the plant. The key element of this subsystem is the cold water pipe (CWP) which, due to its massive size and operating environment, will present a significant design problem. The successful performance of the CWP is of crucial importance to the overall OTEC system; the pipe itself is considered the most critical part of the entire operation.

Because of the importance of the CWP, a project for the analysis and design of CWP's was begun in the fall of 1978. The goals of this project were to study a variety of concepts for delivering cold water to an OTEC plant, to analyze and rank these concepts based on their relative cost and risk, and to develop preliminary designs for those concepts which seemed most promising.

The characteristics of the platform from which the CWP will be suspended and the environmental conditions at the operational site are both influential in the analysis of the CWP. Two representative platforms and sites were chosen: a spar buoy of a Gibbs and Cox design to be moored at a site off Punta Tuna, Puerto Rico, and a barge designed by APL/Johns Hopkins University, grazing about a site approximately 200 miles east of the coast of Brazil. Sufficient water will be supplied to each plant to generate 40 MWe net power.

In order to perform the required analysis and develop the CWP designs, we at TRW assembled a team of experts to assist us in specific areas. Global Marine Development, Inc., a member of our OTEC team since 1974, is providing us with expertise in naval architecture and ocean operations. Their talents have focused on analysis and design of the joint between the CWP and the hull, and also in development of scenarios and an estimation of deployment loads during installation of the CWP. Hydronautics Inc., an expert in fluid mechanics and structural dynamics,

is assisting in determining CWP loads and in analyzing problems associated with vortex shedding. ETA/FSE, a small partnership of consultants in Atlanta, Georgia and Bellingham, Washington, has assisted us in estimating the mechanical properties and fabrication techniques associated with FRP CWP's. Goodyear Aerospace Corporation has provided us with the properties and analysis of their cable reinforced, ring stiffened elastomer CWP, and Sir Robert McAlpine and Sons, Ltd., of Great Britain has participated in developing designs and costs associated with the construction of large concrete structures.

1.1 COLD WATER PIPE REQUIREMENTS

The requirements for the CWP which were previously established during Task 1 provide the basis for the concept analysis and performance evaluation of Task 2. These requirements define the function of the CWP and quantitize as much as possible the performance which must be achieved by the CWP subsystem. The fundamental requirement for the CWP designs to be developed during this project is:

Deliver approximately 4600 cu ft/sec (2.1 million gal/min) of cold seawater from a depth of about 3000 feet (2500 feet for the grazing plant mode) to an OTEC power subsystem at the ocean surface at minimum cost, subject to the following restrictions/limitations:

- Design life: 30 years minimum
- Increase in temperature of cold water from bottom to top of pipe: less than 1°F (see Appendix A6)
- Operational environment (defined in Appendix A2)
- Survival environment (defined in Appendix A2)
- Platforms: the Gibbs and Cox spar and the APL/JHU barge
- Environmental impact: meet existing or envisioned environmental guidelines during all phases of CWP development and operation
- Safety: meet all existing or envisioned safety regulations (Coast Guard, ABS, OSHA, etc.)
- Design producibility: must be compatible with MEP development schedule.

Additional performance requirements which amplify the fundamental requirement and serve to act as design guidelines are:

- Be capable of being deployed, operated, and maintained and/or replaced at sea
- Withstand loads imposed during transportation/deployment
- Withstand operational loads imposed by:
 - platform interaction
 - sea interaction
 - flow in pipe
- Minimize the loads that the CWP imposes on platform
- Be capable of detaching (and subsequent recovery) or survival while attached to the platform, during the worst loading conditions, whichever mode is most cost-effective.

1.2 GENERAL APPROACH

Our approach was to concentrate on the most promising concepts and on those which were either of general interest or espoused by others (e.g., steel and concrete concepts). Much of our overall attention, therefore, focused on analyzing rigid and compliant wall designs, while stockade (except for the special case of the FRP stockade) and bottom-mounted concepts received less attention. We also concentrated on the "swing" items insofar as cost and risk were concerned, and spent less time on such items as inlet screens that were common to all the pipes and whose effect on system cost and risk was minimal.

Figure 1-1 shows the design methodology used in analyzing each of the concepts. Using the candidate concepts, evaluation criteria, and ranking methodology developed during Task 1, we performed a conceptual design iteration to establish practical CWP configurations from structural as well as fabrication/construction/installation standpoints. We then developed cost estimates and qualitative risk factor assessments to rank candidate concepts for the CWP categories given in the RFP.

Figure 1-1. Work Approach for Task 2, Analysis for Concept Selection

Design efforts were sufficiently detailed to either develop practical design configurations or eliminate candidate concepts. The emphasis was then placed on making relative cost and risk assessments to compare alternatives. We will refine design and fabrication/construction/installation details and cost estimates further during the preliminary design phase for the three concepts selected by NOAA.

1.3 SUMMARY

A total of 67 CWP concepts were initially generated and subjected to a screening process. Of these, 16 were carried through design analysis, costing, and ranking. Figure 1-2 shows the results of the costing analysis for the spar at the Puerto Rico site. If only fabrication/construction is considered, rigid FRP with fixed joints and rigid concrete with articulating joints are the least costly designs. However, when deployment and non-recurring costs are considered, the concrete becomes considerably more expensive than the FRP. The compliant wall pipes of the Goodyear and Bell Aerospace types are somewhat more expensive than the FRP, but still moderate in price. Most expensive are the steel pipes, both with and without articulating joints.

Results of our concept risk analysis are shown in Figure 1-3. Average ratings run from 3 (high risk) to 8 (low risk) showing that all unfeasible concepts (ratings 1 to 2) had been eliminated during the screening process. However, all concepts analyzed involve some degree of extrapolation and risk, so there were no rankings in the 9 to 10 range. Of all the concepts considered, rigid FRP (sandwich wall) was judged to have the lowest risk. A compliant wall pipe (cable reinforced elastomer, ring stiffened) and rigid steel were the next lowest in risk. Articulated joint design (both concrete and steel) and all bottom-mounted concepts were highest in risk. Finally, polyethylene concepts were found to be higher in risk than FRP in all common CWP categories. Our cost and risk analysis is described in Section 5.

Figure 1-2. Best Estimate Costs (Puerto Rico - Spar)

Figure 1-3. Concept Risk Evaluation Averaged Risks

Figure 1-4 depicts the results of our CWP ranking. Two types of rigid FRP CWP's, sandwich wall and solid wall, are ranked 1 and 2 because of both low cost and low risk. A compliant pipe of the Goodyear type was ranked number 3, offering an interesting alternative to the first two choices. The most promising steel and concrete configurations ranked low because of the risk associated with the articulating joints for concrete and the very high cost of a steel pipe with rigid joints. Finally, Figure 1-5 shows a ranking of CWP design approaches by category and by material. A complete description of the development of our ranking methodology, along with the results of the ranking exercise, is presented in Section 6.

RANKING	CATEGORY	MATERIAL	DESIGN FEATURES
1	RIGID	FRP/(BALSA)	SANDWICH WALL
2	RIGID	FRP	SOLID WALL
3	COMPLIANT	ELASTOMER	CABLE REINFORCED, RING STIFFENED (GOOD YEAR)
4	STOCKADE	FRP	-
5	MULTIPLE*	FRP	-
6	COMPLIANT	ELASTOMER	PRESSURIZED DOUBLE WALL (BELL)
7	MULTIPLE	POLYETHYLENE	-
8	RIGID	CONCRETE	ARTICULATED JOINTS
9	RIGID	STEEL	WELDED JOINTS
10	BOTTOM MOUNTED	FRP	SANDWICH WALL

*SPECIAL CATEGORY CONSIDERED BY TRW/GMDI - NOT IN RFP

Figure 1-4. CWP Concept Ranking Cost and Risk

RANK	CATEGORY	RANK	MATERIAL
1	RIGID	1	FRP
2	COMPLIANT	2	ELASTOMER
3	STOCKADE	3	POLYETHYLENE*
4	MULTIPLE PIPES	4	CONCRETE
5	RIGID-ARTICULATED JOINTS	5	STEEL
6	BOTTOM MOUNTED		

*APPLICABLE TO MULTIPLE PIPES ONLY

Figure 1-5. Best CWP Design Approaches Cost and Risk

2. CONCEPT DEVELOPMENT, DESCRIPTION, AND SCREENING

Development of concepts for CWP's began in 1974 during TRW's first OTEC contract when we, along with our team member Global Marine Development, Inc., investigated FRP, steel, concrete, and elastomer pipes for delivery of cold water to a 100 MWe OTEC power plant. The number of concepts envisioned at that time was greatly expanded during our proposal effort for this present contract in the spring of 1978 when a total of 67 concepts was generated. Four types of pipes:

- rigid
- compliant
- stockade
- bottom-mounted

were suggested by the RFP along with several potential materials:

- reinforced concrete
- steel
- plastic (FRP, polyethylene, etc.)
- elastomer (cable reinforced, pressurized, etc.)

For these pipes, a variety of wall construction techniques were envisioned including such options as solid wall, sandwich, and hollow wall. Pipes both with and without articulating joints were considered as were concepts whose weight was supported by either the wall itself or by a central support member. Figure 2-1 depicts the four generic types of CWP described in the RFP along with some of the options described above.

Figure 2-1. Various Cold Water Pipe Configurations

Because it was not possible to analyze the 67 concepts in sufficient detail to enable us to determine a cost and risk for each, we established a screening process. A board of senior technical personnel representing a variety of disciplines was convened, and the merits and weaknesses of each of the concepts were discussed at length. Worksheets, including concept descriptions and comments for each of the configurations are included in Appendix K. The approach used to eliminate concepts from further analysis used one of three criteria:

- overwhelming technical or schedule problems
- design/material incompatibility
- duplication

After two half-day meetings of the board, sixteen concepts remained for further analysis. These are depicted by wall types in Figure 2-2. The remainder of Task 2 consisted of performing hydrodynamic, dynamic, and structural analyses of these concepts; development of fabrication/installation plans; and generation of cost and risk data for the ranking process. These subtasks are described in Sections 3, 4, and 5 respectively.

3. DESIGN ANALYSIS

3.1 MATERIALS

The primary material candidates for the cold water pipe walls are steel, fiberglass reinforced plastic (FRP), reinforced and prestressed concrete, polyethylene, and reinforced elastomer.

3.1.1 Materials Selection

The selection process consisted of weighing the materials' advantages and disadvantages in terms of the design concepts for the pipe. The criteria were:

- Long-term mechanical and physical properties and degradation modes in the ocean environment over the 30-year service life of the pipe.
- Cost, availability, and the cost of material fabrication.

From the candidate list, specific material types were chosen as the most promising and practical.

3.1.2 Design Allowables

After specific material types were chosen, detailed mechanical and physical properties were obtained to specify the design allowables necessary for the dynamic and structural analysis of the CWP design concepts. Government, industrial, or other material specifications were used as sources for the material properties when available. When such specifications were not available, the properties were taken from the literature or from the results of industrial or government testing.

3.1.3 Steel

Various investigators consider steel a strong candidate for the CWP structural material. Applications include the construction of the whole pipe (wall, stiffeners, joints) and such components as stiffening rings for the elastomeric concepts or rebar for the concrete concepts. The CWP must be designed to last 30 years in seawater under a variety of static and dynamic load conditions, be deployable by achievable techniques, and be reasonable in cost. The use of

structural steel has many advantages in large structures, such as the CWP which include:

- Excellent availability in a variety of shapes, sizes, and alloys.
- Low cost compared to other metallic materials.
- Good cost projections possible because fabrication and raw material costs are well known.
- Easy fabrication in existing facilities by a variety of methods: welding, mechanical fastening, forming by forging, rolling, etc., are state of the art and specific alloys have been developed for selected fabrication methods.
- A history of exposure to marine environments, both submerged and above water.
- The attributes of good specific strengths (of the order of 230×10^3 in. to 350×10^3 in.) and specific stiffnesses (106×10^3 in.) coupled with good elongation.
- Flexibility in the CWP design approach because of the variety of alloys, forms, and fabrication methods available.

Using steel also has disadvantages, and these must be translated into a risk either to the overall cost or to performance. The important disadvantages are listed below.

- Steel must be protected from corrosion in a marine environment.
- High-strength steels, which would be used in fasteners, are susceptible to stress corrosion cracking (SCC) in seawater.
- The more inexpensive steels may possess low notch toughness, especially in the weld areas.
- Steel has a low corrosion-fatigue strength, especially in the weld area. This again requires that the pipe be protected.
- No active or passive corrosion protection systems are known to have been designed for the 30-year maintenance-free life requirement for the CWP.
- No cathodic protection system has been used to depths of 3000 feet for a life approaching 30 years.

- A steel CWP is very heavy, requiring large, expensive handling and deployment equipment.
- The cost-per-unit volume is higher than for nonmetallic materials such as the elastomers or fiberglass-reinforced plastics.

3.1.3.1 Candidate Steels and Their Mechanical Properties

The following steels are considered as candidates for the CWP applications:

- ASME SA 36: a general-purpose, low-cost, weldable structural steel widely used in heavy construction.
- ASTM A 441: a high-strength, low-alloy steel used in welded bridge and building structures. Its atmospheric corrosion resistance is better than the structural carbon steels.
- ASME SA516 Grade 70: a mild steel with good notch toughness widely used in pressure vessel design.
- ASME SA 537 Class 1: a heat-treated, weldable plate steel used in heavy construction.
- HY 80: a high-strength, high notch-toughness steel developed by the Navy for use in hulls.
- HY 100: a higher strength version of HY 80 that also has good notch toughness.
- AISI 4330 V Mod: a high-strength steel with fair notch toughness that has been used in threaded form in seawater applications. It is a candidate for use as high-strength nuts and bolts.

The mechanical properties of these materials (base metals) are shown in Table 3.1-1. Because the prime loads that the CWP must survive are cyclic in nature and are generated by wave motion and vortex shedding, the fatigue strength is of prime importance. The welds will be the weakest link in the CWP design since they will have inferior notch toughness and fatigue strength. It has been conclusively shown (Reference 3-1) that the high-cycle fatigue strength of welded joints in structural steel is independent of the static strength of the parent metal in the range of yield strength of 30 to 110. Gurney and Maddox performed a rigorous statistical analysis of air-fatigue data of welded steel joints and recommended design strengths based on specific types of welds; these data are shown in Table 3.1-2.

Table 3.1-1. Candidate Steel Room Temperature Properties

STEEL*	FTU (KSI)	FTY (KSI) (MIN)	ELONGATION (% IN 2 IN.)	DENSITY (LB/IN. ³)	CHARPY V-NOTCH** (FT-LB)
ASME SA36	58 TO 80	36	23	0.283	10 TO 30
ASTM A441	62 (MIN)	41	19	0.283	20 TO 50
ASME SA516 GR 70	70 TO 90	38	21	0.283	40 TO 50
ASME 537 CLASS 1	70 TO 90	50	22	0.283	30 TO 50
HY 80	100 (MIN)	80	20	0.283	50
HY 100	120 (MIN)	100	18	0.283	50
AISI 4330 V MOD	235 (180)	196 - 205 (165)	12	0.283	20

*COMPOSITIONS:

ASME SA36: Fe - 0.29C - 1.0 Mn - (0.2 Cu)

ASTM A441: Fe - 0.22C - 1.05 Mn - 0.2 Cu - 0.02V

ASME SA516 GR 70: Fe - 0.28C - 1.05 Mn

ASME SA537 CL 1: Fe - 0.24C - 1.05 Mn - 0.3S:

HY 80: Fe - 0.18C - 0.25 Mn - 2.6 Ni - 1.4 Cr - 0.4 Mo

HY 100: Fe - 0.20C - 0.25 Mn - 2.85 Ni - 1.4 Cr - 0.4 Mo

AISI 4330 V MOD: Fe - 0.306 - 0.90 Mn - 1.8 Ni - 0.8 Cr - 0.4 Mo - 0.07 V

**30° TO 100°F, TYPICAL VALUES

Typical notch toughness (Charpy V-notch) of the candidate steels is also shown in Table 3.1-1. It is not clear whether the CWP will be classified as part of the power system piping system or as part of the hull. If it carries the first classification, the Department of Transportation/Coast Guard (DOT/USCG) regulations will apply; if it is classified as part of the hull, the American Bureau of Ships (ABS) rules will apply.

- DOT/USCG 115 Marine Engineering Regulations specify in Section 54.05-6 that "Toughness tests shall be conducted at temperatures not warmer than 10°F below the service temperature . . ." and the minimum allowed is 20 ft/lb of energy when tested by the Charpy V-notch method (Reference 3-2). The DOT/USCG regulations also stipulate (Section 54.05-17) that this toughness requirement is applicable to weld metal and the heat affected zone (HAZ) of the parent metal in weld assemblies.

Table 3.1-2. Recommended Air-Fatigue Design Stresses for Steel Weldments

TYPE OF JOINT	MEAN FATIGUE STRENGTH (KSI)		MEAN FATIGUE STRENGTH MINUS TWO STANDARD DEVIATIONS (KSI)	
	10 ⁵	2x10 ⁶	10 ⁵	2x10 ⁶
I CONTINUOUS AUTOMATIC LONGITUDINAL BUTT WELDS	47.7	25.4	43.6	23.4
II CONTINUOUS AUTOMATIC LONGITUDINAL FILLET WELDS	46.9	30.3	36.7	15.8
III CONTINUOUS MANUAL LONGITUDINAL FILLET WELDS TRANSVERSE MANUAL BUTT WELDS	46.0	16.3	33.2	11.8
IV STUD SHEAR CONNECTORS STIFFENER TO WEB JOINTS	39.2	13.9	29.8	10.6
V SUBMERGED ARC BUTT WELDS BUTT WELDS MADE ON PERMANENT BACKING BAR K BUTT WELDS (CRUCIFORM JOINTS) INTERMITTENT LONGITUDINAL FILLET WELDS TRANSVERSE NON-LOAD-CARRYING FILLET WELDS TRANSVERSE LOAD-CARRYING WELDS (CRUCIFORM JOINT) TRANSVERSE LOAD-CARRYING FILLET WELDS (LAP JOINT)	33.5	11.9	22.3	7.9
VI NON-LOAD-CARRYING WELDS ON A PLATE EDGE LONGITUDINAL LOAD-CARRYING FILLET WELDS BEAMS WITH WELDED COVER PLATES	23.4	8.3	18.0	6.4

- ABS rules require a 20 ft/lb energy absorption by the Charpy V-notch test method at 0°C (32°F) for Class "B" steels (Reference 3-3). Class B steels are those used in structural members containing relatively high stresses. The ABS rules are not clear as to whether welds are included in this notch toughness requirement.
- Det Norske Veritas regulations for offshore structures, Sections 6.1.5.10 and 6.1.5.11, specify the same notch toughness requirements as the DOT/USCG 115 rules (Reference 3-4).

It is not known which, if any, of these regulations apply to the OTEC CWP. If they do apply, the notch toughness requirement will be 20 ft/lb energy (Charpy V-notch) when tested at a temperature of 30°F. The criteria used by OTEC must be based on safety, insurability, and the structural integrity of the finished structure.

While the base metal toughnesses shown in Table 3.1-1 would generally meet the 20 ft/lb requirement, the weldments would have to be carefully evaluated. The weld notch toughness requirement can be met for the candidate steels, except possibly by ASME SA 36, by using multipass welding. This type of welding is more expensive than the single-pass welds assumed for construction of the CWP, and a cost uncertainty must be allowed in pricing the steel concepts.

3.1.3.2 Degradation in Seawater

Steel is prone to a variety of corrosion attack mechanisms in seawater, including general corrosion, crevice corrosion, corrosion fatigue, fretting corrosion, pitting corrosion, and SCC. Galvanic couples must also be avoided because seawater is an efficient electrolyte. Some typical values for corrosion rates and corrosion fatigue strengths are given in Table 3.1-3 (from References 3-5, 3-6, and 3-7).

Table 3.1-3. Typical Corrosion Data for Steel Immersed in Seawater

<u>GENERAL CORROSION RATE</u>	5 MPY
<u>PITTING CORROSION RATE</u>	
BASE METAL	3 TO 10 MPY
WELDS	3 TO 40 MPY
<u>CORROSION FATIGUE STRENGTH AT 10⁷ CYCLES</u>	
BASE METAL	5 KSI CARBON STEELS 15 KSI HY 80
WELDS	APPROXIMATELY 80 PERCENT REDUCTION IN AIR-FATIGUE STRENGTH

For similar test conditions, the general corrosion rate of steel is independent of composition to a remarkable degree. Variations in the pitting rates have been noted for different types of steel, however, and additions of chromium to steel have been used to retard pitting. As would be expected, SCC susceptibility is strongly dependent on composition and strength level.

Other parameters affecting corrosion behavior are presented qualitatively in Table 3.1-4. La Que (Reference 3-5) has estimated that the general corrosion rate doubles for every 20°F rise in temperature; this rate is found to increase five-fold when the water velocity is increased from zero to 6.5 fps. The corrosion rate is proportional to the oxygen content in the seawater. Chlorine is known to disrupt the oxide film which forms on the metal surface thus increasing the diffusion of corrodents. The pH has little effect except for values above nine, at which point general corrosion is retarded. The presence of cold work in the metal (and, thus, residual stresses) increase susceptibility to SCC and corrosion fatigue.

The presence of static stress is required for SCC and has a small negative effect on corrosion fatigue strength. It has also been reported to have a small effect on general corrosion rates. The presence of alternating stresses is obviously required for corrosion fatigue and fretting fatigue.

There are other complications that can influence corrosion behavior, especially on large, complex structures: the presence of stray currents can greatly accelerate localized corrosion rates, as can galvanic couples with more noble metals. Extremely high velocities, due to sharp turns in flow streams, can cause erosion-corrosion, and cavitation corrosion is a well-known attack mechanism in pumps and propellers.

While data for steel behavior are quite complete on general and pitting corrosion and a good understanding of the other mechanisms exists, each structure and exposure condition must be carefully analyzed to ensure that excessive attack does not occur.

3.1.3.3 Corrosion Control

Because of both the multiple corrosion mechanisms that can attack the CWP and the extremely severe performance requirements, measures must be taken to control corrosion and prevent failure. Good design practice that can mitigate corrosion attack to a degree includes avoiding galvanic couples (e.g., at the screen attachment), crevices, moving (especially rubbing) parts, sharp transitions in metal cross section, and sharp changes in seawater flow direction. Beyond this, one or more of three approaches must be employed: providing a corrosion allowance, providing

Table 3.1-4. Effect of Environment on Corrosion Performance of Steels in Seawater

CORROSION MECHANISM	TEMP.	WATER VELOCITY	OXYGEN CONTENT	PARAMETER			STATIC STRESS	ALTERNATING STRESS
				CHLORINE CONC.	pH	COLD WORK IN STEEL		
GENERAL CORROSION	↑	↑	↑	↑	0	0	↑SMALL	0
PITTING CORROSION	↑	?	↑	↑	0	0	0	0
CREVICE CORROSION	↑	?	↑	↑	0	0	0	0
GALVANIC CORROSION	↑	↑	↑	↑	0	0	0	0
FRETTING CORROSION	↑	0	↑	↑	0	0	↑	↑
CORROSION FATIGUE	↑	?	↑	↑	0	↑	↑SMALL	↑
SCC	0(?)	0	↑	↑	0	↑	↑	0

KEY: ↑ INCREASE IN VALUE OF PARAMETER INCREASES RATE OF DEGRADATION.
 ↓ INCREASE IN VALUE OF PARAMETER DECREASES RATE OF DEGRADATION.
 0 INCREASE IN VALUE OF PARAMETER HAS NO EFFECT ON RATE OF DEGRADATION.
 ? INCREASE IN VALUE OF PARAMETER HAS UNKNOWN EFFECT ON RATE OF DEGRADATION.

a passive protection system, and providing an active protection system. Cathodic protection can prevent or minimize general corrosion, pitting corrosion, crevice corrosion, and corrosion-fatigue, but it will not prevent SCC. The three approaches are discussed separately below.

a. Corrosion Allowance. A preliminary analysis was performed to determine the corrosion allowance of a steel CWP designed to last 30 years (Reference 3-8). The oxygen concentration as a function of depth for both the Pacific and Puerto Rico sites, velocity inside the pipe, and the mean water temperature were taken into account. Based on a zero current acting on the outer surface of the pipe, a corrosion allowance of 0.265 in. was calculated. Using the same approach but taking into account a one knot (1.67 fps) current and a 90°F surface temperature, the corrosion allowance would be approximately 0.39 in. near the surface and 0.29 in. for most of the length of the pipe.

b. Passive Protection System. There are two types of passive corrosion control systems that can be used: coatings or cathodic protection using sacrificial anodes. The two are often used in combination to optimize cost and minimize maintenance. Several coatings have been used in marine applications, including those based on epoxy, coal-tar epoxy, and neoprene elastomer. Of these, the only coating that has supporting data for a 30-year life in seawater is neoprene (Reference 3-9). Neoprene can be applied by spray or paint techniques; the typical application is a 0.060-in. thickness applied in multiple layers to eliminate pinholes or other through defects. Neoprene is somewhat more expensive than the epoxy or coal-tar epoxy coatings and, like other elastomeric coatings, is not used with cathodic protection because hydrogen generated at the steel surface can cause debonds.

Epoxy coatings are normally applied in thicknesses of 0.015 in. to 0.030 in. and are typically designed for a service life of 5 to 10 years. Coal-tar epoxies are more compliant than the epoxies and are widely used for service life of 7 to 10 years. Coal-tar epoxies are frequently used in conjunction with cathodic protection systems, which reduce the number and cost of anodes and the cost of the overall system. A coating thickness of approximately 0.016 in. to 0.018 in. is used for this type of service.

Passive cathodic protection systems utilize sacrificial anodes, which are usually made of aluminum or zinc. The anodes are placed to provide maximum coverage for the protected area. Private communication with an expert in cathodic protection system design was made to assess the applicability of cathodic protection to a steel CWP. His opinion was that the pipe exterior could be protected by hull-mounted anodes, but that the interior presents a special problem because of the high velocity of seawater. It may be necessary to distribute anodes along the inside surface of the pipe; if so, maintenance of the protection system becomes a major problem. Sacrificial systems, however, normally require less maintenance than impressed current systems. The anode distribution and maintenance problems become even more severe for some of the built-up concepts that contain multiple flooded channels running the length of the pipe. Cathodic protection systems are currently designed for 10 to 20 years life.

Table 3.1-5 presents typical properties for three commonly used galvanic (sacrificial) anode materials (Reference 3-10). If a protective current density of 10 ma/ft^2 for a 30-year design life and a protected area of $570,000 \text{ ft}^2$ is assumed, a required weight of aluminum anodes of 1.2 million lb is obtained. The use of a coating can substantially reduce this requirement.

Table 3.1-5. Galvanic-Anode Characteristics and Properties

ANODE PROPERTY	MAGNESIUM MIL-A-21412	ZINC MIL-A-18001	ALUMINUM (A1 - 0.5Zn - 0.05 Hg)
TYPICAL POTENTIAL (VOLTS VS Ag/AgCl)	1.55	1.04	1.06
CURRENT CAPACITY (AMPERE HRS/LB)	500 (AT 50% EFFICIENCY)	368 (AT 99% EFFICIENCY)	1250 (AT 93% EFFICIENCY)
NOMINAL ANODE CONSUMPTION (LB/AMPERE YEAR)	17	24	7
DENSITY (LB/IN ³)	0.063	0.258	0.099

c. Active Protection System. Impressed current cathodic protection is used either alone or, more often, with an organic coating. Anodes are nonconsumable and are typically made of platinum-clad or platinized titanium, graphite, silicon-iron, lead-silver, or lead-platinum. A direct current voltage and current is impressed by use of a dc motor or rectified alternating current. Electrical connections to the anodes must be sealed to prevent corrosive failure and, normally, dielectric shields are provided immediately between the anode and protected surface to increase the throw area of each anode. Depths of 200 ft are considered deep applications and maintenance becomes a difficult problem; the 3000-ft depth creates a problem that is beyond state-of-the-art practice. An impressed current system will require extensive development before it can be used on the OTEC CWP. Associated problems include the effect of the high-velocity seawater inside the pipe on connections, mounting brackets, and ground wires. Failures have occurred on offshore structures due to current-induced vibration (vortex shedding) (Reference 3-11).

When using cathodic protection, either sacrificial or impressed current, problems associated with overprotection must be considered. Materials susceptible to SCC are particularly critical since overprotection causes hydrogen generation which can trigger SCC failure. High-strength bolted structures are therefore probably not compatible with cathodic protection for the CWP. The problem is even more complex since coatings over bolts can fail, allowing a crevice condition that can also greatly accelerate SCC (Reference 3-12).

3.1.3.4 Recommendations

The following recommendations are made for assessing steel pipe concepts during the conceptual design tradeoffs.

1. Carry forward four of the steels shown in Table 3.1.1 (ASME SA 36, ASME SA 516 GR 70, HY 80, HY 100) as candidates. These cover the spectrum from a low-cost steel to a very high-strength marine steel. ASTM A 441 is intermediate between SA 36 and SA 516 GR 70 in cost and quality, and ASME SA 537 Class 1 requires heat treatment, which seems impractical for a structure the size of the CWP.
2. Use all-welded construction. Avoid bolts, especially those made of high-strength steel (yield strength greater than 130 KSI), because of the high risk of SCC.

3. Since a corrosion protection system is required and the task of designing one to last 30 years is very difficult, use a combination approach: sacrificial anode cathodic protection in conjunction with a coal-tar epoxy coating. In addition, to guard against possible failure, an 0.1-in. corrosion allowance should be applied to all thicknesses. This combination approach is justified by the critical nature of the CWP to the success of the OTEC system.

3.1.4 Fiberglass Reinforced Plastic (FRP)

Fiberglass reinforced plastics have long been used in marine applications. The U.S. Navy has been using FRP for small boat construction since 1947 (Reference 3-13) and, in 1965, studied the 11-year old FRP fairwater from the submarine U.S.S. Halfbeak and found it still in good condition. In their 1971 evaluation of FRP boats for material durability, Graner and Della-Rocca concluded that FRP provided the features essential for a long-lasting, durable hull. These, of course, are the same features (inertness to the seawater, light weight, high strength) needed for the CWP. Easy fabrication of FRP cylinders by filament winding (see Figure 3.1-1) also enables the pipe designer to tailor an efficient, lightweight, load-carrying structure by placing the glass fiber reinforcement in the principal stress directions without incurring a large cost penalty.

Fiberglass reinforced polyester or vinyl ester resin was the selected material for the various FRP CWP concepts. They are the least costly, and the most widely available of the resins used in the FRP industry. Ease of processing the catalytically cured polyester and vinyl esters during fabrication of large FRP structures dictated their choice over epoxy resins. They also have good physical, mechanical, and corrosion-resistant properties for this application.

Figure 3.1-1. Reinforced Plastic and Elastomer Filament Winding

3.1.4.1 Mechanical Properties

There are two types of mechanical properties of concern for the CWP: short- and long-term.

a. Short-Term. Since most commercial FRP pipe is designed for static loading (i.e., sewer pipe) that is primarily circumferential, the first task was finding realistic short-term, or initial, tensile strength and modulus values for large-diameter, filament-wound FRP pipe. The highest fiber loadings of these pipes are in the hoop direction, and are achieved by the use of helical winds of angles between 55 and 65 degrees as measured from the axial direction of the pipe. No data currently available is specifically keyed to large, axially loaded, FRP structures. The mechanical properties of the circumferential windings of these cylinders is fairly well understood since the fibers are uniformly tensioned (because of the constant curvature of the cylinder) and remain

so after curing into a rigid composite. The mechanical properties of the longitudinal windings are not well characterized: unless special procedures are used, the longitudinal fibers may not be uniformly tensioned and the composite after cure cannot efficiently utilize all the fibers in the axial direction when loaded. One method of tensioning the axial fibers uniformly during the winding process is to pull each longitudinal glass-ply down into a recess at the ends of the mandrel by means of a circumferential wrap (see Figure 3.1-2). Another possibility is to use unidirectional pultrusions for the longitudinal plies. In the pultrusion process the fiber is held under tension during cure and can be efficiently loaded as a composite (see Figure 3.1-3).

Figure 3.1-2. End Tensioning Method for Longitudinal Filament-Wound Plies

Figure 3.1-3. Typical Pultrusion Process

The tensile strength of the bare E-glass rovings ranges from 150,000 to 200,000 psi. For 50 percent-by-volume glass fiber unidirectional laminates, therefore, the ultimate tensile strength in that direction should fall into the 75,000 to 100,000 psi range. The tensile modulus should be 6.3×10^6 psi based on the E-glass modulus of 10.5×10^6 psi. In all these cases, the strength and modulus of the resin matrix is neglected. The short-term mechanical properties used in our FRP analyses were based on the following theoretical considerations: for a biaxial laminate ($0^\circ/90^\circ$) construction with a glass-to-resin volume ratio of 1:1, which corresponds to a 60 percent-by-weight glass fiber to 40 percent-by-weight resin content, the room temperature mechanical properties are:

	Ultimate Tensile Strength (psi)	Tensile Modulus (psi)	Interlaminar Shear Strength (psi)	Poisson's Ratio
Circumferential Ply (Hoop)	90,000	6×10^6	1,500	0.22 (Composite Value)
Longitudinal Ply (Axial)	90,000	6×10^6	1,500	

A 2:1 (Axial-to-Hoop) composite laminate would therefore have the following mechanical properties:

Longitudinal (Axial)	60,000	4×10^6	1,500	0.22
Circumferential (Hoop)	30,000	2×10^6	1,500	0.22

b. Long-Term. The two main concerns for the long-term mechanical properties of FRP in the CWP are static fatigue, or stress rupture, and dynamic fatigue. To these must be added the influence of seawater immersion and the hydrostatic effect of pressure at depth. For long-term static loading, FRP structures are designed to a strain limit; this is primarily based on work done by Isham and others (References 3-14 through 3-18). They found that the life of an FRP tank could be predicted on the basis of strain; pipe and sewer manufacturers refer to the strain-to-first-crack, or the weep point. One manufacturer, for example, has found that for their FRP sewer pipe, this corresponds to about 0.009 in./in. strain. For design they use a 5:1 safety factor for loading, or 0.0018 in./in. maximum strain; for transient or occasional loadings, a strain of 0.003 in./in. which represents a safety factor of about 3:1. The 0.1 percent strain limit for long-term static loads is currently an industry-wide standard for FRP structures.

The long-term cyclic loading of FRP has been the subject of a great deal of investigation. In their report on the mechanism of fatigue in FRP, Broutman and Sahu (Reference 3-19) studied a biaxial laminate (0° , 90°) and determined the S-N curve shown in Figure 3.1-4. Note that as the fatigue stress level drops to the 40 percent strength level, the curve begins to level out and, at a stress level of approximately 40 percent of the ultimate, 26,000 psi for their laminate, the life was 10^7 cycles. They attribute this to the fact that the stress-strain curve for this laminate was linear up to 20,000 psi, which corresponds to a strain of 0.4 to 0.5 percent. From that point and up to 28,000 psi, the curve was non-linear to failure. Other investigators have found a similar trend; however, an air endurance limit for FRP has never been conclusively demonstrated.

Figure 3.1-4. S-N Curve for Cross-Ply Glass Fiber Reinforced Plastic Laminate ($R = 0.05$)

3.1.4.2 Degradation in Seawater

Fiberglass reinforced plastic may be subject to mechanical properties degradation on exposure to water. The degree of degradation of different FRP composites varies considerably, depending on the component materials employed and the quality of the finished laminate. While earlier types of materials suffered significantly from water degradation, improvements have made current FRP materials relatively insensitive to this problem.

a. Strength. The effects of extended seawater immersion on the strength of a number of earlier types of glass-reinforced polyester laminates are shown in Figure 3.1-5. Although the materials were affected to different extents, the curves are similar in that there is a period of relatively rapid degradation, followed by a period of diminished degradation, and finally a flattening of the curve indicating essentially no further degradation with time of exposure. This response is typical of FRP materials. In the case of stiffness modulus, the effect of seawater immersion is less clearly defined. In general, however, it has been found that the stiffness of FRP composites is much less sensitive to seawater degradation than is strength.

Figure 3.1-5. Compressive Strength Retention of FRP Laminates After Seawater Immersion

b. Stress Rupture. While earlier investigations indicated that the stress rupture or static fatigue resistance of FRP degraded with exposure to seawater, Mandell and McGarry (Reference 3-20) found that water immersion tends to decelerate the growth rate of cracks under static loading. Little data on the dependence of static fatigue resistance of FRP in seawater is available: since 1963, FRP pipe manufacturers have used a steady pressure rating test that calculates the 100,000 hour (11.4 year) steady stress life. The average degradation in strength in 100,000 hours is 65 percent. These tests are run with saltwater as the pressurant. Extrapolating the data to 30 years (approximately 263,000 hours), the strength degradation becomes approximately 69 percent. The failure or end-point in this test is when the pipe begins to leak or weep and, again, this is not directly applicable to the CWP. By extrapolating glass mat laminate data from Mandell and McGarry on static tensile strength versus time-to-failure, the 30-year degradation in strength under dry static loading is 77 percent. Extrapolating the filament-wound laminate curve (Figure 3.1-6) shown by N. Fried (Reference 3-21) on interlaminar shear stress-rupture in seawater, the interlaminar shear strength degradation after 30 years is approximately 65 percent.

Figure 3.1-6. Interlaminar Shear Stress-Rupture Curve 2:1 Filament Wound FRP

c. Fatigue. Data are available on both filament-wound and glass-laminate FRP composites for long-term cyclic loading in seawater. Figure 3.1-7 shows S-N curves for both composites as a function of the percent of the original ultimate strength. The curve for the glass mat laminate was based on tensile strength data. The corresponding dry fatigue curve for the mat composite is also shown. A straight line extrapolation of the air and seawater mat laminate curve data gives a tensile strength degradation of approximately 80 percent for 10^7 cycles. Note the negligible difference between the two.

Figure 3.1-7. S-N Curves Linear Approximations

d. Hydrostatic Pressure. The effect on flexural, compressive, and interlaminar shear strength of exposure of polyester laminates to seawater at high hydrostatic pressure is shown in Table 3.1-6. It is evident from the data extant on the exposure of FRP to seawater at high pressure that the material is no more adversely affected than by exposure at atmospheric pressure. It is also worth noting in the table that the modulus of elasticity was unaffected; however, the void content of the FRP can have some effect on the pressure sensitivity of the composite. While the data shown were based on tests with high-quality, low-void content laminates, the differences exhibited by poorer quality laminates will generally be relatively small.

Table 3.1-6. Effects of Deep Ocean Exposure on Glass Fiber Reinforced Polyester Laminates

- EXPOSURE DEPTH: 5700 FT

PROPERTY	INITIAL	4 MONTHS	35 MONTHS
COMPRESSIVE STRENGTH (PSI)	33,000	31,000	33,500
COMPRESSIVE MODULUS (PSI)	2.72×10^6	2.61×10^6	2.90×10^6
FLEXURAL STRENGTH (PSI)	38,800	39,500	38,100
FLEXURAL MODULUS (PSI)	2.6×10^6	2.59×10^6	2.50×10^6
INTERLAMINAR SHEAR STRENGTH (PSI)	1810	1680	1880

Our approach, then, has been to use theoretical values for FRP short-term mechanical properties, and to modify those values to fit the long-term mechanical properties requirements by degrading them by the seawater immersion values shown. The worst-case degradation is fatigue in seawater where the tensile strength, assuming a conservative straight line extrapolation, drops 80 percent after 10^7 cycles. For the CWP design, then, the end-of-life strength is 20 percent of the original ultimate. For the 60/40 laminate, the axial ply strength then drops from 90,000 psi to 18,000 psi using this approach. The composite values drop accordingly.

3.1.5 Reinforced and Prestressed Concrete

The concrete mix selected for the CWP application must be designed and selected to fulfill its intended function economically during the required life of the structure. In addition to the usual requirements for all concrete, concrete used in a marine environment must be selected with special attention to durability.

The following sections discuss the general failure mechanisms of concrete subjected to the marine environment and an evaluation of the APL/JHU proposed lightweight concrete mix design. The lightweight mix design was assessed by C. Richardson and others (Reference 3-22) in 1979 and, in the findings of their study summarized in Subsection 3.1.5.2, they suggest additional study be given to a heavier weight concrete.

The design strength properties of the concrete, prestressing wire, and reinforcing steel are presented in Table 3.1-7. The alternate mix design is the medium weight concrete. The concrete strength allowables for various stress conditions (e.g., compressive, bending, and shear stresses) are, in general, based on the 28-day compressive strength of the mix design. The design strength allowables for prestressed and reinforced concrete were based on the American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete, ACI-318-77, and related supplements. Review of Det Norske Veritas, Fédération Internationale de la Précontrainte, and ACI rules for concrete sea structures (References 3-4, 3-23, 3-24) indicated that the ACI Building Code Requirements were consistent with industry practice for the design of concrete structures exposed to the ocean environment. Tensile cracking of concrete under design service loads will be avoided.

Table 3.1-7. Concrete CWP Material Properties

CONCRETE	DRY DENSITY (PCF)	COMPRESSIVE STRENGTH (PSI)	RUPTURE MODULUS (PSI)	POISSON RATIO	YOUNG'S MODULUS (PSI)
LIGHT-WEIGHT CONCRETE	85	4000 TO 4500	500	0.2	1 TO 1.2 X 10 ⁶
MEDIUM-WEIGHT CONCRETE	100	5500	550	0.2	2.4 X 10 ⁶

- PRESTRESSING WIRE
 - ASTM A416 – GRADE 270
 - YOUNG'S MODULUS – 28 X 10⁶ PSI
 - ALL TENDONS SOLID GROUTED WITHIN SEMI-RIGID METAL DUCTS
- REINFORCING STEEL
 - ASTM A615 – GRADE 40
 - YOUNG'S MODULUS – 29 X 10⁶ PSI
- STRUCTURAL STEEL
 - SEE STRUCTURAL STEEL CWP CONCEPTS FOR HY80 STEEL

3.1.5.1 Degradation in Seawater

The durability of the concrete material in seawater is of concern especially for new and unproven mix designs. Historically, concrete vessels have shown high durability and required relatively little maintenance. Concrete hulls submerged for up to 60 years have sustained little seawater degradation. Current practices for the estimation of annual maintenance costs for concrete-fixed oil production facilities for North Sea Operations are conservatively taken as one percent of the cost of the construction and installation of the structures as opposed to steel oil production facilities at five percent (Reference 3-25).

Nevertheless, the marine environment constitutes a very severe test of concrete's durability. Concrete itself is subject to chemical attack and disruption, and reinforced concrete is subject to electrochemical corrosion as well. Abrasion by moving sediments and attack by marine organisms also contribute to the deterioration of marine concrete.

3.1.5.2 Corrosion Control

a. Abrasion. All sea structures are subject to abrasion from moving water carrying sand or silt particles; this is of primary importance near the surface. Studies have shown that structural lightweight concrete is subject to plucking of coarse aggregate particles along its surface. This can be minimized by thorough curing (to increase the bond between paste and aggregate) and by providing wire mesh or closely-spaced reinforcement near the surface. Concrete abrasion resistance can be improved by polymer impregnation or by exterior coating of dense polyurethane or epoxy applied without solvent. These coatings are reported to greatly reduce the adherence of marine growth to the exterior surfaces of the concrete (Reference 3-26).

b. Biofouling. All sea structures are subject to fouling by marine organisms. Concrete with compressive strengths above 3000 psi have shown good resistance to marine organisms. Concrete subjected to the sea environment will experience less fouling from marine growth than steel because of its alkalinity. Boring marine organisms (clams, sea urchins, and rock-boring mollusks) deposit acids which dissolve the cement and are most aggressive on porous concrete (Reference 3-23). A hard, dense concrete surface will generally provide adequate protection (Reference 3-27). Since

an OTEC concrete CWP would be deployed in tropical/subtropical waters favored by these organisms, further testing of the proposed lightweight concrete mix design should be performed to assess their attacks. Concrete blocks removed from the Los Angeles breakwater after over 67 years of exposure to the sea still showed the original form marks after the barnacles, etc., were removed; they showed little or no corrosive attack by the secreted acids of the marine organisms (Reference 3-24).

c. Chemical Corrosion. Seawater contains sulphates and chlorides that attack concrete in a manner similar to the sulphate attacks that occur when concrete structures are buried in soils with a high groundwater table.

The chemical products of the reaction when concrete is attacked by a sulphate solution are gypsum and calcium sulphotoaluminate. These products have a much greater volume than the compounds they replace, so that the reactions with the sulphates will lead to expansion and disruption of the basic concrete bonding of materials. The chemical attack proceeds more rapidly in warm seawater and is of major significance in tropical and subtropical waters. Concrete's resistance to sulphate attack depends on its impermeability: it is always permeable to a degree, so seawater tends to flow either by pressure differential or by capillary attraction into it (Reference 3-27). Impermeability is the best protection; to be impermeable, the concrete must have a low water/cement ratio (i.e., dense mix design). Use of cement containing moderate or low amounts of tri-calcium aluminate (less than 8 percent) also improves the concrete sulphate resistance (Reference 3-4).

Steam curing also improves the resistance of concrete to sulphate attack (Reference 3-28). This applies both to concretes made sulphate resistant and to ordinary Portland cements, since the improvement is due to change of C_3AH_6 into a less reactive phase and to the removal of $C_a(OH)_2$ by the reaction with silica during the curing process. The addition of calcium chloride to the mix will lead to a reduced resistance to sulphate attack, whatever type of cement is used.

The deleterious action of sulphates in seawater on the exposed concrete is similar to that of sulphate-bearing ground water. Attacks in seawater, however, are not accompanied by the normal concrete expansion encountered

by concrete subjected to sulphate-bearing ground water. The absence of expansion is mainly due to the presence in the seawater of a large quantity of chlorides that inhibit the expansion. Gypsum and calcium sulphoaluminate are more soluble in a chloride solution than in ground water, and are thus leached out by the seawater.

d. Corrosion of Embedded Reinforcing Steel. The most serious challenge to concrete structures in marine environments is the corrosion of embedded reinforcing steel. Salts absorption establishes anodic and cathodic areas in reinforced concrete and the resulting electrolytic action leads to an accumulation of the corrosion products on the steel with a consequent rupture of the surrounding concrete. The effects of seawater are more severe on reinforced concrete, therefore, than on plain concrete. It is essential to provide a sufficient cover to the reinforcement and to use dense, impermeable concrete. A minimum cement content of 500 lb/yd³ or a water/cement ratio of not more than 0.40 to 0.45 is recommended by the American Concrete Institute for the design and construction of fixed offshore concrete structures (Reference 3-24). The water/cement ratio is believed to be the vital factor. However, the cement content is also important, primarily because a high content makes it possible to achieve full compaction of mixes with low water/cement ratios. A well-compacted concrete and good workmanship, especially in the construction of joints, are of vital importance. The type of cement used is of secondary importance with aluminous sulphate-resisting, Portland blast-furnace, and Portland-possolana cements all giving good results (Reference 3-28).

The standard practice in the design of concrete structures for marine environments is to use compressive prestress to overcome excessive tensile stresses. Control of tensile stresses is important because excessive concrete cracking can lead to corrosion of the rebars and prestressing tendons. The additional cost of such prestressing does not significantly increase the cost of the concrete structure. Cracks in concrete have long been blamed for accelerating corrosion of reinforcing steel and have often been called a primary cause. It is generally believed that cracks narrower than 0.01 in. (hairline cracks) will not contribute to corrosion. Well-distributed reinforcement, low water-cement ratio, good finish, and proper curing are all procedures required for crack prevention.

The general effect of submergence on concrete is not well known, but it may be positive. Salt water slightly reduces the effective concrete strength, eliminates drying shrinkage, and reduces creep. The tensile strength of saturated concrete is slightly greater than for dry concrete. The tensile strength of dry, lightweight concrete is lower than that for dry, normal weight concrete of the same compressive strength; when moist or saturated it is approximately the same. These phenomena are both related to the favorable effect of absorption in reversing shrinkage and micro-cracking at the aggregate/paste interface that occurs during curing.

The advantages and disadvantages of concrete materials for CWP applications are presented in Table 3.1-8. The physical properties of the concrete materials were summarized in Table 3.1-7. These materials properties, coupled with the physical properties, dictate the nuances of the concrete CWP that must be studied during design. The basic technology for construction of a concrete CWP exists, but it will require additional testing to determine the final mix design to ensure the required life of the CWP.

Table 3.1-8. Concrete CWP Material

ADVANTAGES	DISADVANTAGES
<p>LOW COST OF MATERIALS: \$800/CUBIC YARD VERSUS \$3240/TON STEEL</p> <p>WIDE MATERIAL AND SEMI-SKILLED LABOR AVAILABILITY RESULT IN LOW COSTS AND IMPROVED CONSTRUCTION-DEPLOYMENT SCHEDULES</p> <p>ABILITY TO TAILOR REINFORCEMENT (PRESTRESS) TO LOAD</p> <p>ABILITY TO MOLD INTO COMPLEX SHAPES AND DOUBLE CURVATURE CONFIGURATIONS LEADING TO OPTIMUM STRUCTURAL DESIGNS</p> <p>DURABILITY AND REDUCED MAINTENANCE</p> <p>FATIGUE RESISTANCE</p> <p>GOOD THERMAL INSULATING QUALITIES (THERMAL CONDUCTIVITY ONE-SIXTH THAT OF STEEL)</p>	<p>LOW TENSILE RESISTANCE IN DIRECT TENSION, FLEXURE, AND DIAGONAL TENSION (SHEAR), ($f_T = 0.1 f'_c$)</p> <p>HIGH-WEIGHT TO STRENGTH RATIO</p> <p>LIMITED DUCTILITY ($\mu \leq 2.0$)</p> <p>HANDLING HEAVY SECTIONS DURING DEPLOYMENT</p> <p>SPECIAL PREPARATION NEEDED TO ENSURE WATER TIGHTNESS AT JOINT</p>

3.1.5.3 Concrete Mix Design

The two lightweight concrete mix design properties, as proposed by Applied Physics Laboratory/Johns Hopkins University (APL/JHU) for OTEC CWP application, are summarized in Table 3.1-9. The data were obtained in a private conversation from Mr. A. E. Fiorato, of the Portland Cement Association, on 24 January 1979. It had been indicated by APL/JHU at the CWP workshop (held in January 1979) that OTEC mix "C" would be proposed for additional studies for CWP applications.

The aggregate proposed is the expanded clay type, trade name LIVLITE, and has the following size graduation unit weight properties:

Coarse	321 lb/ft ³
Medium	381 lb/ft ³
Fine	391 lb/ft ³

Table 3.1-9. Light Weight Concrete Mix Designs (APL/JHU)

MATERIAL	OTEC MIX 'B' LBS/YD ³	OTEC MIX 'C' LBS/YD ³
CEMENT TYPE 1	540	611
FLY ASH	86	97
COARSE AGGREGATE	363	350
MEDIUM AGGREGATE	388	416
FINE AGGREGATE	312	314
WATER	312	346
WATER REDUCING AGENT	3.1	3.5
AIR ENTRAINING AGENT	2.6	2.7
PERCENT AIR ENTRAINED	8	5
DENSITIES OBTAINED (CONDITION)		
AIR DRIED	69	77
SATURATED *	89	92
SATURATED **	76	80

* 4-IN. DIAMETER BY 8-IN. HIGH CYLINDERS IMMERSSED AT 1400 PSI
PRESSURE (EQUIVALENT TO 3000-FT DEPTH)

** 4-IN. DIAMETER BY 8-IN. HIGH CYLINDERS IMMERSSED AT
ZERO PRESSURE

Design strengths (28-day compressive strength) of the two mix designs were given as 3000 and 4000 psi, respectively, and an additional 500 psi allowed only to give a target strength (i.e., concrete strength used for design) for the mixes. The design strengths were determined from tests performed on standard specimens: 6-in. diameter by 12-in. high cylinders. The allowance of 500 psi for the project design strength (characteristics strength) of 4500 psi indicated that a very high level of quality control would be required during fabrication/construction site.

The characteristic strength of these mixes appears to be less than the 4500 psi stated by APL/JHU.

A characteristic strength of 4500 psi on cylinders can be taken as equivalent to 5600 psi on standard 6-in. cubes. The British Standard Codes of Practice 110 recommends that in a case such as this where little data are available, to establish a target strength for a test mix an allowance of 2175 psi in excess of the required characteristic strength should be taken to give a target mean strength for the mix design. The American Concrete Institute has similar standards.

The code will allow a smaller margin, but not less than 1090 psi, when the required characteristic strength approaches the maximum possible strength of the concrete for the initial mix design. This is equivalent to requiring the target mean strength for mix designs of 5340 psi on 6-in. diameter test cylinders.

The expanded clay aggregate proposed by APL appears to be at its design strength limit. Extensive field quality controls will be required during CWP fabrication to ensure that concrete strength properties are obtained. The limited number of laboratory specimens tested does not guarantee that the concrete strengths could be duplicated in the field.

Schedule limitations have not allowed TRW and McAlpine to actually prepare and test alternative mixes. However, it is felt that a stronger aggregate than an expanded clay would be needed to give the required target mean strength as described above. This would inevitably mean an increase in the density of the concrete. Since it is the saturated density of the concrete that is critical in the CWP design, however, the percentage

variation in density using different aggregates is less than would be the case with air-dried densities.

Various types of lightweight concrete aggregates have been examined and their typical properties are listed in Table 3.1-10. Additional strength properties and reported tested data for the lightweight aggregate concretes can be obtained from Reference 3-26.

Table 3.1-10. Light-Weight Aggregate Concrete

MATERIAL	AGGREGATE BULK DENSITY (LB/FT ³)	MAXIMUM CONCRETE STRENGTH (PSI)
EXPANDED CLAY	FINE 40	3500
	COARSE 21	
EXPANDED SHALE	FINE 60	4650
	MEDIUM 44	
EXPANDED SLATE	FINE 54	5225
	COARSE 35	
BLAST FURNACE SLAB	FINE 70	5225
	COARSE 50	
SINTERED PFA	FINE 65	6375
	MEDIUM 52	
SINTERED COLLIERY SHALE	FINE 60	5225
	COARSE 44	

The target mean strength of 5340 psi is the maximum strength reasonably achievable with all aggregates except the sintered PFA. For a target strength of 5225 psi, concrete using this aggregate would typically have an air-dried density of 106 lb/ft³ (pcf) and a saturated density of 117 pcf. With an increase in cement content, however, a reduction in fines and from 8 to 10 percent air entraining this saturated density could be reduced to 106 pcf. Some additional refinement of the mix (e.g., cement content and accelerated steam curing) could reduce the density further with minimal loss of concrete strength. Mix designs using aggregates lighter than the sintered PFA may yield the required strength, but high cement contents and natural sand fines would be required and would result in saturated densities of the same order.

The recommendation for 8 to 10 percent air entrainment should only be used with great caution and after extensive tests because experience has indicated that in this strength range of concrete, there is a greater loss of air than expected and a considerable variation in air content. Care should be exercised in the selection of the final aggregates, as an abundance of fines may result in a breakdown of the material during mixing. Additional testing will be required of the final, lightweight concrete mix design.

McAlpine has recommended that the lightweight (85 pcf mix design) be analyzed in greater detail before use of the mix design for OTEC CWP applications. The mix design would be a high-risk item, requiring extensive research and development, for the CWP concept evaluation matrix. A medium weight concrete with a saturated density of between 100 to 106 pcf would be more readily obtainable. The target mean strength of the mix design would be 5340 psi on the standard cylinders, which would be equivalent to a design-allowable specified concrete strength of 4500 psi.

3.1.6 Polyethylene

High density polyethylene (HDPE) has found increasing acceptance as a large diameter piping material for marine applications. The combination of low density and inertness, along with toughness, abrasion resistance, and relative ease of fabrication, make it a natural choice for water lines, sewer outfalls, etc. When properly compounded with appropriate agents, HDPE's resistance to degradation in the marine environment envisioned for the CWP is probably limitless.

The main concern with HDPE for the CWP is its structural behavior. Polyethylene (PE) is a viscoelastic material whose mechanical properties depend on time. The viscoelastic properties can be divided into two general types: creep, which is the strain in time when the material is held at a constant stress, and stress relaxation, which is the change in stress with time when the material is held at a constant strain.

3.1.6.1 Static Loading

Creep varies so greatly with time that it is customary to plot it against log time so straight lines can be obtained. There are deviations at very long and very short times, however, and the same is true for stress relaxation. The property of creep and its corollary, relaxation, complicate the establishment of modulus values for polyethylene. Modulus is the ratio of stress to strain and, because of this time dependence, it is not constant. For the creep modulus, therefore, strain increases with time under a constant load; for the relaxation modulus, stress diminishes with time under constant strain. Young's modulus, traditionally used for elastic materials, is, for polyethylene, the instantaneous value found only over short time durations or low stress levels where creep is not induced. A typical constant strain curve is shown in Figure 3.1-8: when a constant load is applied, a primary strain (e_A), which is elastic in nature, results over a short period of time (t_1). The rate of strain then slows and progresses at a steady rate over a period ($t_2 - t_1$). Finally, when a critical level of strain, e_B , has been reached, the rate of strain again accelerates and rupture occurs after a further, unpredictable, short time period ($t_3 - t_2$) following elongation to the rupture strain level, e_C .

Figure 3.1-8. Polyethylene Stress-Creep Relationship

For the HDPE we intend to use for our concepts, the critical strain level (ϵ_p) is between 5 and 7 percent strain depending on the imposed load and the duration of the load. The critical strain then is the limit for predictable life. The HDPE CWP will then be designed on the basis of this limit-state method (Reference 3-28). For a 50-year life, the factored design strain turns out to be 3 percent, which provides a safety margin of about two. Figure 3.1-8 shows a plot of constant stress curves on a graph of strain versus time under stress. For a 3 percent strain level, the stress duration exceeds 1×10^6 hours (114 years) at a stress level of 700 psi; this provides the safety factor of two for a 50-year design.

On the basis of 3 percent strain, the apparent creep modulus has been calculated for this material as a function of temperature (Figure 3.1-9). To find the "instantaneous modulus" one must then extrapolate down to short time durations (seconds) to find the modulus. Table 3.1-11 shows the apparent modulus shown by a high density polyethylene as a function of loading rate.

Figure 3.1-9. Apparent Modulus Versus Time and Temperature, High-Density Polyethylene

Table 3.1-11. Apparent Modulus at Varying Loading Speeds

LOADING TIME (SEC)	DYANMIC MODULUS, PSI (73°F)
10^{-4}	276,000
10^{-2}	245,000
1	122,000
100	87,000

3.1.6.2 Cyclic Loading

If the duration of each loading pulse is long, then creep and recovery analysis is indicated for HDPE. Figure 3.1-10 shows the allowable stress duration versus fluctuating stress superimposed over a steady stress for HDPE pipe. For intermittent stressing of this nature, the recovery time must at least equal the stress duration. If the duration of each loading pulse is short, however, the end result of such a cyclic loading process may be fatigue failure. For HDPE, cyclic loadings at frequencies above 100 cpm can result in severe autogenous heating due to internal friction (hysteresis). The failure for non-isothermal, high-frequency cycling of HDPE is by softening or necking; for low-frequency isothermal cycling, the failure is by fairly abrupt fracture (Reference 3-30). Little reliable data is available in the literature because of this problem on long-term fatigue endurance of HDPE; however, a recent study by Sauer, Foden, and Morrow of Rutgers University has shed some light on the fatigue behavior of polyethylene (Reference 3-31). These investigators, using isothermal low-frequency 100 cpm cyclic loading, found increases in molecular weight lead to increases in fatigue life. Figure 3.1-11 shows the dramatic increase in fatigue endurance as the molecular weight of polyethylene went from 5,000 (low) to 50,000 (medium). For the high-molecular weight polyethylenes (160,000 to 2,100,000), the endurance limit exceeded 10^7 cycles. It should also be noted that the 3000 psi stress was very close to the yield strength for the medium- and high-molecular weight polyethylenes. The investigators caution that the data is too limited to predict the endurance limit of the higher molecular weight polyethylenes and believe that the endurance limit of these materials may only realize a modest increase compared to that shown between 5,000 and 50,000 molecular weight PE.

Figure 3.1-10. Allowable Stress Duration Versus Fluctuating Stress

Figure 3.1-11. Endurance Limit Dependence on Molecular Weight of Polyethylene

3.1.7 Reinforced Elastomer

Reinforced elastomers exhibit properties that are unique for composites: high-strength filamentary materials such as nylon, polyester, or steel in combination with a flexible, high-elongation elastomer matrix are used in structures that must resist high-tensile loads, but must also deform easily or with little resistance to compressive or flexural loads (hoses and automotive tires).

Elastomers have the following characteristics that make them an ideal matrix material for this application:

- High deformation without breaking
- Rapid and repeated recovery from deformation
- Large deformations in proportion to original dimensions
- Large deformations at relatively low stress levels
- Increased stiffness with increased deformation.

Goodyear proposed a CWP concept that used a steel wire rope reinforced neoprene composite based on their standard conveyor belt construction, with the difference that the neoprene cover would be thick on both sides instead of just one. The material has already been used for sonar domes, so extensive experience in the marine environment exists. With the proper amount of neoprene covering the steel cables, the material shows good resistance to degradation by seawater immersion (Figure 3.1-12). Neoprene itself has particularly good resistance to water and seawater immersion: properly compounded neoprenes will normally exhibit a 10 percent-by-volume swell during long immersion in seawater. The loss in tensile strength is about 13 percent after 12 months exposure, which is probably the equilibrium value. In combination with steel reinforcements, the neoprene in thicknesses greater than 0.060 in. probably acts as a semi-permeable membrane preventing oxygen and pH-reducing salts from diffusing with the water into the interface. The oxygen-free neutral water that does contact the steel wire is not corrosive.

Figure 3.1-12. Ocean Aging of Sonar Dome Material

3.1.7.1 Physical and Mechanical Properties

The typical physical and mechanical property values for the neoprene matrix used for this construction are as follows:

- Specific Gravity 1.37
- Hardness (Shore A) 50-60
- Tensile Strength (psi) 2500
- Elongation Ultimate (Percent) 750
- Shear Modulus 107
- 30 Percent Deflection (psi)

3.2 HYDRODYNAMIC LOADING ANALYSIS

To successfully design a cold water pipe to perform in the ocean environment for 30 years, hydrodynamic loads due to surface waves and currents, internal pipe flow, and internal waves must be estimated. Most current experimental data cannot be directly applied to the OTEC CWP design because the parameters governing the scaling of the experimental data do not match OTEC conditions. No data exists for Reynolds numbers, $Re = 10^7$, in wavy flows with superimposed current flows. The CWP hydrodynamic design loads must nevertheless be estimated if OTEC is to be built, and engineering judgment must be relied upon to successfully extrapolate the experimental data.

The flow past bluff bodies is complicated by the fact that boundary layer separation occurs and vorticity generated in the boundary layer organizes in the wake and travels downstream in some regular fashion. For a rigid cylinder in steady flow, this organization usually takes the form of the well-known Karman vortex street and leads to periodic lift and drag forces on the body. Periodic vortex shedding from fixed cylinders has been the subject of many experimental and theoretical investigations and values of lift, drag, and shedding frequency are fairly well characterized as a function of roughness and Reynolds number.

For cylinders that respond dynamically to the flow, suitable oscillations of the body cause significant changes in the wake structure and in the hydrodynamic forces acting on the cylinder. Resonant oscillations of the body can be excited normal or parallel to the incident flow direction. For crossflow oscillations, the shedding frequency of the vortices can change from its normal value to a value near one of the natural frequencies of the structure. This "lock-on" occurs when the cylinder natural frequency is within 20 to 30 percent of the usual Strouhal shedding frequency. Under lock-on conditions, the fluid forces are significantly amplified over the fixed cylinder case.

In-line resonant conditions may also occur. In air, the in-line component of the periodic force, at twice the Strouhal frequency, is an order of magnitude less than the crossflow component and has usually been ignored. In water, however, because of the greater density and hence larger fluid forces, an in-line resonant condition may be excited.

The previous discussion pertains to lightly damped structures subject to currents alone. The situation is further complicated when waves are present; the lift may be amplified as much as 4.5 times the fixed cylinder value.

Pressure losses inside the pipe relative to the outside pressure at the same depth arise from viscous losses, density differences, and plant-induced transients. For thin wall CWP designs, pipe buckling can occur because of the pressure differential across the wall. Because of the time-dependent nature of the external flow, transverse dynamic buckling or fatigue failure must also be considered.

Internal waves act in the ocean at depths below the influence of surface waves. The energy density of these waves falls off at the Brunt-Vaisala frequency,

$$N^2(Z) = -g \left(\frac{1}{\rho} + \frac{\partial \rho}{\partial Z} + \frac{g}{c^2} \right)$$

where the density ρ and sound speed c depend on the depth Z . The large temperature (and associated density) gradients found in the OTEC preferred sites may result in Brunt-Vaisala frequencies comparable with the lowest CWP response frequencies.

3.2.1 Approach

Because most of the available experimental data cannot be directly scaled to OTEC conditions, the approach taken in the present work was to extrapolate the appropriate data based on the similarity of particular flow regimes. Data obtained for Reynolds numbers high enough to ensure

turbulent separation should be applicable for OTEC CWP designs at higher, but similarly turbulent, separation Reynolds numbers.

Because of the complexity of the bluff body separated flow phenomena, investigators have concentrated on examining the sensitivity of the flow to a limited number of parameters at a time. For OTEC, several effects (i.e., waves, current shear, density gradient, pipe motion, and internal pipe flow) impact the hydrodynamic loads. The relative importance of each and their interaction must be assessed, and the experimental data must be scaled and applied in appropriate proportion.

For preliminary concept evaluation and selection the hydrodynamic forces have been estimated by considering the flow about 30-ft diameter circular pipes 2500- and 3000-ft long.

3.2.1.1 OTEC Scaling Parameters

The basic parameters that ensure similarity between scale model tests and the OTEC prototype CWP are the Reynolds number, $Re = UD/\nu$, Keulegan-Carpenter number, $K = V_m T/D$, and Richardson number

$$R_i = \frac{g}{\rho} \frac{\partial \rho}{\partial Z} \bigg/ \left(\frac{dU}{dZ} \right)^2$$

where V_m is the maximum wave particle velocity and U is the current velocity. The parameter $\beta = D/U_m dU/dZ$ is useful in examining the effect of current shear on vortex shedding; U_m is the mean current velocity.

For the stationary mode plant the NOAA OTEC Demonstration Plant Environmental Package (Reference 3-32) specifies a current profile normalized by surface velocity. The Reynolds number regime for OTEC is then approximately as given in Reference 3-33 (see Figure 3.2-1). For Punta Tuna the maximum surface current is 1.42 m/sec and under normal conditons $U_s = 0.62$ m/sec.

Figure 3.2-1. Flow Reynolds Number Profile for a Nine Meter Diameter CWP

The maximum wave height expected to be encountered is given in Reference 3-32 as $H_{\max} = 79.6$ ft for the Punta Tuna area under hurricane conditions. This corresponds to a Keulegan-Carpenter number of

$$K_{\max} = \frac{V_m H_{\max}}{D} = \frac{\pi H_{\max}}{D} = 8.34$$

Under normal conditions for significant wave heights, $H_s = 12$ ft (Sea State 5, $U_s = 24.1$ Knots, Reference 3-32) $K = 1.256$.

The range of Richardson number is plotted in Figure 3.2-2 for the Punta Tuna area. The current shear parameter for the extreme current condition varies from about $\beta = 0.033$ at 100 to 200 meters, to $\beta = 0.015$ for depths greater than 450 meters (see Figure 3.2-3).

Figure 3.2-2. Richardson Number Dependence on Depth at the Punta Tuna Area for Both Normal and Extreme Velocity Distributions

Figure 3.2-3. Extreme Current Profile

3.2.2 External Flow - Vortex Shedding

3.2.2.1 Rigid Cylinder

For smooth cylinders in cross flow pressure distributions, the drag and lift forces and vortex shedding frequency have been measured for a range of Reynolds numbers by several investigators (References 3-33 through 3-37). Their findings show that the fluid dynamic forces and wake characteristics depend largely on the state of the boundary layer near the separation point. For laminar separation ($Re \leq 10^5$), values of drag, lift, and shedding frequency are well correlated. As the Reynolds number increases and is critically close to boundary layer transition, laminar separation occurs. The flow re-attaches, however, forming a separation bubble and then turbulently separates again; this critical range is seen as a sudden drop in the drag (Figure 3.2-4). Also, in the critical range, definite vortex shedding frequencies may disappear because the boundary layer is unstable and a regular shedding mechanism is difficult to set up.

Figure 3.2-4. Characteristic Curve of the Drag Coefficient Versus Reynolds Number

In the supercritical range ($4 \times 10^5 < Re < 3 \times 10^6$), transition to turbulence occurs in the boundary layer and turbulent separation occurs. Regular shedding frequencies are not usually observed in this range. With

further increases in the Reynolds number, the point at which transition occurs in the boundary layer moves away from separation upstream toward the forward stagnation point. At the highest Reynolds numbers tested to date, $Re \approx 10^7$, transition is very near the forward stagnation point, the wake is again organized and definite shedding frequencies are observed.

Reynolds numbers for a 10/40 MWe OTEC plant fall in the range of 10^6 to greater than 10^7 will be encountered (Figure 3.2-1). For a smooth cylinder in normal flow, good estimates of the hydrodynamic forces can probably be obtained by extrapolating existing test data. Boundary layer characteristics should not change significantly. Also, the character of the turbulent wake probably will not change enough to alter the forces on the cylinder.

OTEC pipes will generally have some roughness and be inclined to the flow. For small inclinations to the flow (<15 deg), forces can be adequately modeled by characterizing the boundary layer using a cross flow Reynolds number, $Re_n = Re \cdot \cos \phi$, where ϕ is the inclination angle.

It is known (Reference 3-38 and 3-39) that increasing roughness will lower the critical Reynolds number, thereby promoting early transition. For OTEC, however, the effect of roughness at high Reynolds numbers ($>10^6$) when the boundary layer is already fully turbulent must be considered. Szechenyi (Reference 3-39) claims that C_L , C_D , and $S = nD/U$, where S is the Strouhal number and n the shedding frequency, may be correlated if those parameters are plotted against a roughness Reynolds number, $Re_\delta = Re_d \cdot \delta/D$, where δ is the effective diameter of the roughness elements (glass spheres were glued onto the cylinder to represent roughness in Reference 3-39). Other investigators (Reference 3-40) claim the drag increases directly with increasing δ/D and that δ/D is the correlation parameter that should be used.

For roughness characteristic of the ocean environment ($\delta/D \sim 1 \times 10^{-3}$) the data collected by SAI (Reference 3-33) was used to provide baseline rigid cylinder values of drag, lift, and Strouhal number, $CD_0 = 0.95$, $CL_{0_{RMS}} = 0.27$, $S_0 = 0.3$.

3.2.2.2 Dynamic Cylinder

For cylinders which respond dynamically to the flow, suitable transverse oscillations cause significant changes in the wake structure and in the hydrodynamic forces acting on the cylinder. Experiments have been conducted by several investigators where the cylinder is forced to oscillate transversely to the flow under specific conditions. When the oscillation frequency is within about 20 to 30 percent of the natural shedding frequency, lock-on occurs and vortices are shed at the cylinder frequency. Under lock-on conditions, the fixed cylinder values of lift and drag are significantly amplified. Self-excited resonant conditions are reproducible with an externally excited cylinder when the test conditions are carefully simulated. However, significant self-excited oscillations are not to be expected at cylinder natural frequencies greater than the natural shedding frequency. Self-excitation would be expected only under conditions where energy is fed to the cylinder (Reference 3-41). Maximum values of lift and drag amplification occur when the cylinder natural frequency is equal to the natural shedding frequency.

Values of lift amplification were obtained by SAI using a wake oscillator model similar to that of Iwan (Reference 3-42). The maximum amplification was about $(C_L/C_{L0}) = 2.6$. The maximum drag amplification from data collected by Griffin (Reference 3-43) is about $C_D/C_{D0} = 1.8$.

For cylinders in oscillating flows (waves) Sparpkaya (Reference 3-44) has shown that the lift force (normal to the wave direction) may be even larger than the lock-on amplified current lift force. The values of lift and drag obtained experimentally by Sparpkaya in a U-shaped water tunnel with harmonically oscillating flow and no current are presented in Reference 3-44 as a function of Keulegan-Carpenter number and Reynolds number.

The variation of Keulegan-Carpenter number $K = \frac{V_m T}{D}$ with depth is presented in Figure 3.2-5 for extreme environmental conditions at the Puerto Rico and Brazil sites. Since the CWP will be attached to the platform at some 60 to 220 feet below the surface, the maximum value of K encountered will be about 5.

Figure 3.2-5. Variation of Keulegan-Carpenter Number with Depth, Extreme Conditions

Sarpkaya's values of lift coefficient:

$$C_L(\max) = \frac{\text{maximum peak of the transverse force}}{0.5 \rho D L V_m^2}$$

and

$$C_L(\text{rms}) = \frac{\text{rms value of the transverse force}}{0.5 \rho D L V_m^2}$$

where V_m = maximum particle velocity, are replotted in Figure 3.2-6. At $K \approx 5$ for the largest Reynolds number investigated, $Re_m = V_m D / \nu = 21,000$, Sarpkaya found $C_L(\max) = 0.63$ and $C_{L_{rms}} = 0.44$.

At this Reynolds number other investigators have measured amplitudes of C_L ranging from 0.45 to 0.9 on smooth fixed cylinders in steady flow. For high transitional Reynolds numbers, lift coefficients of $0.1 < C_{L_{rms}} < 0.2$ have been measured on smooth cylinders and values of $C_{L_{rms}} = 0.27$ have been obtained for roughened cylinders in steady flow.

Figure 3.2-6. Lift Coefficient As a Function of Keulegan-Carpenter Number ($Re_m = K\beta$)

The period of vortex shedding ($S = D/T_s V_m$) measured by Sarpkaya is related to the period of the harmonic waves, $T_w = KD/V_m$, as

$$\frac{T_s}{T_w} = \frac{D}{S V_m} \frac{V_m}{KD} = \frac{1}{SK}$$

When the wave period is greater than the natural shedding period ($S = 0.2$ laminar, $S = 0.3$ turbulent separation) vortices will be shed from the cylinder before the flow reverses. When the wave period is less than the shedding period, vortices will be forming behind the cylinder but will not be completely shed when the flow changes direction. If $S = 0.2$, then $T_s/T_w = 5/K$, and for $K \lesssim 5$ asymmetry may not develop and lift will be very small. Taking $K_{\text{max}} = 5$ then from

$$K = \frac{V_m T}{D} = \frac{\pi H}{D}$$

$H_{\max} = 48$ ft for the OTEC CWP. For environmental conditions where severe wave heights are not encountered, lift forces are expected to be small and should be bounded by the self-excited steady current results. The variation of drag coefficient with Keulegan-Carpenter number is presented in Figure 3.2-7. For $K = 5$, $C_d = D/0.5 \rho V_m^2 DL = 1.2$, which is approximately equal to the steady drag value measured on rough cylinders.

Figure 3.2-7. C_d Versus K for $\beta = 4072$ and $k/D = 1/360$.

The variation of Strouhal number with K is presented in Figure 3.2-8. Strouhal number exhibits a strong relationship on K ($S = 0.63 \log K + 0.83$) for $5 < K < 10$ and then becomes constant at $St = 0.22$ for $K > 15$.

In most cases the CWP will be subjected to wave action near the surface superimposed with a current flow. Since Keulegan-Carpenter numbers of greater than about $K = 5$ are unlikely to be encountered, the maximum values of lift and drag will be those corresponding to locked-on shedding in uniform or shear current flow. The recommended values of the hydrodynamic coefficients are given in Table 3.2-1. For flows with waves and currents, $U = U_c + V_m$ should be used to compute the forces from the coefficients in the table.

Figure 3.2-8. Strouhal Number Versus K for a Particular Rough Cylinder

Table 3.2-1. CWP Hydrodynamic Coefficients in Current Flow

V_c = CURRENT VELOCITY
(ROUGHNESS $S/D = 1 \times 10^{-3}$)

REGULATE VORTEX SHEDDING		LOCKED-ON VORTEX SHEDDING	
$C_{L_{O_{RMS}}}$	.27	C_L/C_{L_0}	2.6
C_{D_0}	1.14	C_D/C_{D_0}	1.0
S_0	0.3	S	0.3

3.2.2.3 Three-Dimensional Vortex Shedding

The dynamic analysis of the CWP requires not only some knowledge of the magnitude and frequency of the cross sectional lift and drag on the pipe, but also an estimate of the correlation and phase of the vortex shedding along the length of the pipe.

In recent literature, Stansby (Reference 3-45), Mair and Stansby (Reference 3-46), Maull and Young (Reference 3-47), and Korotkin (Reference 3-48) have dealt with the motion of vortices being shed in cells of finite extent in uniform and shear flows. It is well known that oscillation of the cylinder tends to increase the correlation length, and it is reasoned that roughness by means of fixing the separation angle will also increase the correlation length.

For the shear flows that will be encountered in the ocean environment, it is apparent that vortex shedding in cells exist. However, using Stansby's correlation

$$\frac{\Delta Z}{D} \frac{a}{D} = \frac{a}{D} \frac{4.0}{\beta_M} \frac{S_c}{S}$$

where

$$\beta_M = \frac{\lambda D}{U_M}, \quad \lambda = \frac{dU}{dZ}$$

$$S_c = \frac{n_c D}{U_M} = \text{nondimensional cylinder frequency}$$

$$S = \frac{n_s D}{U_M + \lambda (Z - Z_M)} = \text{nondimensional shedding frequency}$$

$$\frac{a}{D} = \text{amplitude of oscillation/cylinder diameter}$$

$$\Delta Z = \text{spanwise extent of locking on}$$

to estimate the cell length for the OTEC pipe results, letting $S_c = S$, and using $\beta = 0.02$, results in $\Delta Z/D = 200 (a/D)$. For deflections on the order of the diameter of the pipe, vortices will supposedly be shed in single cells along the entire length of the pipe.

Stansby's correlation was primarily based on data where the amplitude of the oscillation was about $a/D = 0.06$; using his correlation for larger amplitudes may not be valid.

If, as demonstrated by experiment, the shedding frequency is constant in a cell then

$$S_M = \frac{nD}{U_m} = \text{Const} , U_M = \text{mean flow velocity}$$

and therefore

$$S_L = \frac{nD}{U_L} \neq \text{Const} , U_L = \text{local flow velocity}$$

Local Strouhal number, S_L , for a particular Reynolds number for forced vibration cylinders varies less than about $\Delta S_L = 0.1$. For different flow regimes (laminar separation versus turbulent separation), S_L is found to vary no more than about one-tenth. Assuming the Strouhal invariance holds in shear flow, the cell length can be estimated from

$$S_M = S_L \frac{U_L}{U_M} = S_L \left(1 + \beta \frac{Z - Z_M}{D} \right)$$

If

$$\frac{S_M}{S_{L1}} = 1 + \frac{\beta(Z_1 - Z_M)}{D}$$

$$\frac{S_M}{S_{L2}} = 1 + \frac{\beta(Z_2 - Z_M)}{D}$$

where

S_{L1} = Strouhal number at the lower boundary $Z = Z_1$ of the cell
(same cell $S_m = \text{constant}$)

S_{L2} = Strouhal number at the upper boundary $Z = Z_2$ of the cell

then

$$\frac{\Delta Z}{D} = \frac{S_m}{\beta} \left(\frac{\Delta S_L}{S_{L2} S_{L1}} \right)$$

Now if

$$S_{L2} = S_m + 1/2 \Delta S_L$$

$$S_{L1} = S_m - 1/2 \Delta S_L$$

$$\frac{\Delta Z}{D} = \frac{S_m}{\beta} \left(\frac{\Delta S_L}{S_m^2 - \frac{\Delta S_L^2}{4}} \right)$$

and for

$$\Delta S_L \ll 1$$

$$\boxed{\frac{\Delta Z}{D} = \frac{\Delta S_L}{\beta S_m}}$$

If $S_m = S_0 = 0.3$ for OTEC and $\beta = 0.02$, then $\Delta Z/D \approx 17$ for $(\Delta S_L)_{\max} = 0.1$.

Many experiments have borne out the fact that to maintain regular vortex shedding, certain spacing between vortices exists. This observation is manifested in the imperturbability of the local Strouhal number. From these observations maximum cell lengths of about $20D$ are indicated. Direct measurements of the spanwise coherence of shedding vortices indicates that cell lengths of $10D$ may be more likely (References 3-39 through 3-41).

Figure 3.2-10. Response of CWP at Time of Maximum Stress

The choice of cell distribution along the pipe and the frequency and phase of shedding in each cell may significantly affect the dynamic response of the CWP. For the concept selection phase, two 10D cells were located: one starting at the surface and the other located from 500 to 800 ft down. The frequencies in these cells correspond to different natural frequencies of the pipe. The pipe response at time of maximum stress is shown in Figure 3.2-10.

Our analysis has provided some basis for concept selection; however, lest designs be discarded on these arguments alone, it should be pointed out that the spanwise coherence data that exists is very limited, and it

is non-existent for OTEC Reynolds number. Additional data and further analysis is required for more detailed design analysis.

3.2.3 Internal Waves

The variation of density with depth given in the environmental report (Reference 3-32) can be used to estimate the behavior of the Brunt-Vaisala frequency in the proposed OTEC sites. The distribution of values for the summer profile at the Punta-Tuna area is given in Figure 3.2-11. If the peak values of the Brunt-Vaisala frequency, $N = 0.019$, is greater than a natural mode of a proposed pipe design, then the dynamics simulation needs to include the time dependence of internal wave velocities in the imposed current. It has been found (Reference 3-49), for example,

Figure 3.2-ii. Brunt-Vaisala Frequency Dependence on Depth for the Punta Tuna Area Summer Density Profile

that the particle velocities due to the internal waves can, under some conditions, be of the same order as the current velocity.

The reaction of proposed pipe designs with response frequencies less than the maximum Brunt-Vaisala frequency can be examined by superimposing an approximation to the internal wave velocities on the ambient current. Internal waves with frequencies close to the maximum Brunt-Vaisala frequency are computed by using the small phase speed approximations given in Reference 3-50.

The possible internal wave effect on the CWP should be evaluated in the more detailed phase of the analysis for a CWP design with very low frequency response.

3.2.4 Internal Flow

Consider pipes which deliver flow from a depth of 3000 ft at a rate equivalent to a 30-ft diameter pipe with an average flow speed of 6.5 ft/sec. Pressure losses relative to the outside flow result from the density difference, friction losses in the pipe, and at the intake screens and intake losses. Denote the pressure difference between the outer and inner side of the CWP wall as:

$$\Delta p(\theta, Z) = q(Z) C_p(\theta, Z) + \frac{1}{2} \rho_I U_I^2 + \int_Z^L (\rho_I - \rho(Z)) g dZ \quad (3-1)$$

$$+ \Delta p_S + \Delta p_F(Z)$$

where

$$\Delta p_F(Z) = (L-Z) \frac{\rho_I}{2D} U_I^2 f\left(\frac{\epsilon}{D}, Re\right)$$

and

- L = Pipe length (3000 ft)
- Z = Depth measured from surface
- q(Z) = Dynamic pressure in external flow
- D = Pipe diameter (30 ft)
- ρ_I = Density inside pipe
- $\rho(Z)$ = Density outside pipe
- $C_p(\theta, Z)$ = Pressure coefficient on outside of pipe
- ϵ = Roughness height inside pipe (0.001 ft)
- U_I = Internal flow velocity (6.5 ft/sec)
- f() = Friction factor
- Δp_S = Pressure drop across screens and inlet
- Δp_F = Wall friction pressure loss
- Re = Reynolds number, based on D (1.2×10^7)

Using the Punta Tuna area summer distribution for the density (Reference 3-32), the value of ρ_I is taken as the density at the 3000-ft depth, giving

$$\frac{1}{2} \rho_I U_I^2 = 0.29 \text{ PSI}$$

and

$$\Delta p_F = (3000 \text{ ft} - Z) \times 10^{-4} \frac{\text{PSI}}{\text{FT}}$$

The contribution due to the difference in hydrostatic pressure is denoted Δp_ρ and is tabulated as a function of depth in Table 3.2-2.

Table 3.2-2. Pressure Difference Across CWP Wall Due to Internal Flow Effects (Punta Tuna Area, Summer Profile)

Z (m)	Z (ft)	Δp_F (PSI)	Δp_ρ (PSI)	$\Delta \hat{p}_{it}$ (PSI)
0.0	0	0.29	2.05	2.87
15.2	50	0.29	1.93	2.87
30.5	100	0.28	1.80	2.61
61.0	200	0.27	1.55	2.35
91.5	300	0.26	1.32	2.11
122.0	400	0.25	1.11	1.89
152.4	500	0.24	0.91	1.68
228.7	750	0.22	0.60	1.35
304.9	1000	0.20	0.38	1.11
457.3	1500	0.15	0.11	0.79
609.8	2000	0.10	0.02	0.65
762.2	2500	0.05	0.01	0.59
914.6	3000	-	-	0.53

The pressure drop across the inlet screens and due to the inlet effects are strongly design-dependent. The loss from the inlet and the screens, taken from the Westinghouse report (Reference 3-51), gives a typical:

$$\Delta p_S = 0.24 \text{ PSI}$$

The total of the internal flow effects on the pressure difference across the CWP are denoted as $\Delta \hat{p}$.

where

$$\Delta \hat{p}_{it}(Z) = \Delta p(\theta, Z) - q(Z) C_p(\theta, Z)$$

The values are tabulated in Table 3.2-2 and shown graphically in Figure 3.2-11.

The Brazil area density variations do not result in a pressure difference as large as for the Punta Tuna Area. The worst cases for the pressure difference are given for the summer profiles at the 15°S, 25°W site. Table 3.2-3 contains the tabulated value for these sites. The CWP is assumed to extend to 2,500 ft for these cases. The pressure dependence on depth at this site is included in Figure 3.2-12.

Table 3.2-3. Effect of Density Variation At Brazil Site (15°S, 25°W)

Z (M)	Z (FT)	Δp_F (PSI)	$\Delta p_\rho(Z)$ (PSI)	$\Delta \hat{p}_{it}$ (PSI)
0	0	0.25	0.87	1.65
40	131	0.24	0.71	1.48
100	328	0.22	0.51	1.26
150	492	0.20	0.39	1.12
300	984	0.15	0.16	0.84
400	1312	0.12	0.08	0.73
510	1673	0.08	0.04	0.66
762	2500	-	-	0.53

The values tabulated above do not account for the motion of the pipe or any curvature in the pipe. The centripetal acceleration in a pipe bend does not contribute appreciable pressure changes. To estimate the maximum load on the pipe for a buckling analysis, the steady outer flow contributions and the forces that arise because of pipe accelerations must be included. The water inside the pipe represents a mass which, when accelerated, results in a reaction force of the pipe wall. Fluid

surrounding the pipe will also act as an added mass and must be considered in computing the buckling stress.

Figure 3.2-12. Operational Pressure Drop Across CWP Wall Due to Internal Flow Effects

The force acting on the cylinder in the relative velocity direction is

$$F_x = \frac{D}{2} \int_0^{2\pi} (\sigma_{rr} \cos \theta - \sigma_{r\theta} \sin \theta) d\theta \quad (3-2)$$

The first term in Equation (3-2) represents the normal stress on the body, the second term represents the shear stress. Neglecting shear term, and considering a unit length of cylinder

$$F_x = \frac{D}{2} \int_0^{2\pi} \sigma_{rr} \cos \theta d\theta$$

where

$$\sigma_{rr} = -p + 2\mu \frac{\partial v_r}{\partial r} \quad (3-3)$$

and p is the usual pressure, v_r is the velocity of the fluid radial to the cylinder center.

If the cylinder oscillations are small compared to the diameter the solution to the governing equations of motion is given by Stokes (1851), (Reference 3-52) in terms of the stream function, ψ , which has the form

$$\psi(r, \theta, t) = g(r, t) \sin \theta \quad (3-4)$$

Now since

$$v_r = \frac{1}{r} \frac{\partial \psi}{\partial \theta} \equiv \text{velocity in the radial direction}$$

$$v_\theta = -\frac{\partial \psi}{\partial r} \equiv \text{velocity in the tangential direction}$$

it can be shown using the relation (Equation 3-4) that the second term in Equation (3-3) is of the form

$$2\mu \frac{\partial v_r}{\partial r} = G \cos \theta \quad (3-5)$$

and then

$$F_x = \frac{D}{2} \int_0^{2\pi} -p(\theta) \cos \theta \, d\theta + \frac{D}{2} \int_0^{2\pi} G \cos^2 \theta \, d\theta \quad (3-6)$$

If the second term in Equation (3-6) is equated to the acceleration terms in Morrison's equation

$$F_x = \frac{1}{2} \rho C_D D |v|v + \frac{\rho \pi D^2}{4} (1 + C_m) \frac{\partial v}{\partial t} \quad (3-7)$$

that is,

$$\frac{D}{2} \int_0^{2\pi} G \cos^2 \theta \, d\theta = \frac{\rho \pi D^2}{4} (1 + C_m) \frac{\partial v}{\partial t} \quad (3-8)$$

Then

$$G = \frac{\rho D}{2} (1 + C_m) \frac{\partial v}{\partial t}$$

and the pipe wall stress σ_n (positive outward) due to the difference between the internal flow velocity and the relative external velocity and acceleration is

$$\sigma_n = -\Delta p (\theta Z) + \frac{\rho D}{2} (1 + C_m) \frac{\partial v}{\partial t} \cos \theta$$

If the pipe motion is separated into components along x and y directions then the stress due to velocity and acceleration in the x direction is

$$\left. \begin{aligned} \sigma_n = & -\Delta \hat{p} - \frac{1}{2} \rho_0 \left| \dot{v}_c + \dot{v}_w - \frac{\partial x}{\partial t} \right| \left(\dot{v}_c + \dot{v}_w - \frac{\partial x}{\partial t} \right) C_p(\theta) \\ & + \frac{\rho_0 D}{2} (1 + C_m) \left(a_x - \frac{\partial^2 x}{\partial t^2} \right) \cos \theta \end{aligned} \right\} \quad (3-9)$$

where v_c is the current velocity, v_w the wave velocity and a_x the wave acceleration in the x-direction. The y equation is similar.

The mean values of pressure coefficient $\bar{C}_p(\theta)$ are given in Table 3.2-4, as well as the mean plus one standard deviation and mean plus two standard deviation values. For the buckling analysis it may be assumed that $C_p(2\pi - \theta) = C_p(\theta)$, $0 < \theta < \pi$.

Table 3.2-4. Pressure Coefficient (Reference 3-33)

θ	$\sqrt{C_p'^2}$	$\bar{C}_p$	C_p	
			1σ	2σ
0	0.015	1.0	1.015	1.03
10	0.020	0.91	0.93	0.95
20	0.026	0.63	0.656	0.682
30	0.036	0.13	0.166	0.202
40	0.053	- 0.45	- 0.503	- 0.556
50	0.070	- 0.113	- 1.20	- 1.27
60	0.093	- 1.75	- 1.657	- 1.936
70	0.112	- 2.20	- 2.312	- 2.424
80	0.128	- 2.50	- 2.628	- 2.756
90	0.138	- 2.55	- 2.688	- 2.826
100	0.149	- 2.25	- 2.499	- 2.730
110	0.155	- 1.75	- 1.905	- 2.06
120	0.160	- 0.95	- 1.110	- 1.27
130	0.160	- 0.70	- 0.86	- 1.02
140	0.156	- 0.68	- 0.836	- 0.992
150	0.150	- 0.68	- 0.836	- 0.98
160	- 0.140	- 0.68	- 0.820	- 0.96
170	0.126	- 0.68	- 0.806	- 0.932
180	0.120	- 0.68	- 0.80	- 0.920

$$C_p(\theta) = \bar{C}_p + \eta \sqrt{C_p'^2} \text{ 1}\sigma \text{ VALUE OF PRESSURE COEFFICIENT}$$

Assuming a wave height $H = 8$ ft, $T = 6.5$ sec, and current velocity in the wave direction of 6 ft/sec, the stress on the CWP wall at the surface (where the density difference is greatest) is found from Equation (3-9) as

$$\sigma_n = - 2.87 \text{ psi}$$

$$- \frac{1}{2} \rho_0 (6 + U_w - \frac{\partial x}{\partial t})^2 C_p(\theta)$$

$$+ \frac{\rho_0 D}{2} (1 + C_m) (a_w - \frac{\partial^2 x}{\partial t^2}) \cos \theta$$

If the maximum particle acceleration due to the waves is taken (from Reference 3-53) as,

$$a_w \sim \frac{2\pi^2 H}{T^2} = 3.75 \text{ ft/sec}^2 \quad (U_w = 0)$$

and it is assumed that the pipe acceleration is of the same order, i.e., $\partial^2 x / \partial t^2 \sim 3.75 \text{ ft/sec}^2$, opposes the wave motion, and $\partial x / \partial t = 0$, then an estimate of the stress at $\theta = 0$ can be found as follows:

$$C_p(\theta) = 1.015 \text{ (1}\sigma \text{ value)}$$

$$-\frac{1}{2} \rho_o (U_c + U_w - \frac{\partial x}{\partial t})^2 C_p(\theta) = -\frac{1}{2} \frac{(1.984)}{144} (6)^2 (1.015) = -0.2517 \text{ psi}$$

$$\frac{\rho_o D}{2} (1 + C_m) (a_w - \frac{\partial^2 x}{\partial t^2}) = \frac{1.984(30) (1 + 0.8) [-2 (3.75)]}{2(144)} = -2.79 \text{ psi}$$

so the total normal force per unit area due to internal pressure losses, external separated flow, fluid acceleration, and pipe acceleration is

$$\sigma_n = -5.91 \text{ psi}$$

The value $\sigma_n = -5.91$ represents only an order of magnitude upper bound estimate for wall pressure. For short waves at depths greater than 60 ft, wave acceleration forces would be much less. For long waves the pipe tends to move with the wave motion, hence, $a_w \approx -\partial^2 x / \partial t^2$ and the last term in Equation (3-9) becomes small.

It may be necessary to examine the cross-sectional response (ovalling) of the CWP to the time-dependent loading due to vortex shedding in the detailed analysis.

3.2.4.1 Plant-Induced Loads

Additional buckling (or bursting) loads can arise due to plant-induced transient loads. The present platform plans appear to be designed with the CWP emptying into a sump. This design eliminates the classic water hammer effect. The transient load which results from a pump failure will be denoted as a surge load. The effect on the CWP depends on the sump design. If the ratio of the sump cross-sectional area to the pipe cross-sectional area is denoted as A_R , then for an open-to-the-atmosphere sump, the relationship between the pressure rise at the top of the pipe and the ratio A_R is shown in Figure 3.2-13.

The platform heave effects the pipe's internal flow and pressure in a similar fashion to the surge effect, but the amplitudes are much smaller. However, the pliable pipe designs may need to be examined more closely for response modes of the internal flow.

Figure 3.2-13. Pressure Rise Due to Surge at Pump Failure

3.2.4.2 Compliant Pipes

Compliant pipes mean large values of KD/Et , where K is the bulk modulus of elasticity for sea water, D is the pipe diameter, E is Young's modulus for the pipe material, and t is the pipe wall thickness. The sound speed in the pipe is given by

$$c^2 = \frac{K/\rho}{1 + \frac{KD}{Et}}$$

For compliant pipes this value becomes small enough to allow compressibility effects in the pipe flow to become important.

The increased head loss in the compliant pipe due to the variation in pipe diameter between ring stiffeners can become substantial if the change in diameter occurs rapidly enough to create regions of separated flow at the stiffeners. This may need to be examined in a detailed analysis.

3.2.4.3 Multiple Pipes

The multiple pipe concept for the OTEC CWP assumes that friction losses do not increase significantly for the smaller diameter pipes. As a check on this assumption, the losses are calculated for a 30-ft diameter pipe and a bundle of twenty-five 6-ft diameter pipes. Assume the following values:

ρ	$= 1.99 \text{ slugs/ft}^3$	(density of sea water)
ν	$= 1.66 \times 10^{-5} \text{ ft}^2/\text{sec}$	(viscosity at 40°F)
D_1	$= 30 \text{ ft}$	(diameter of large pipe)
D_2	$= 6 \text{ ft}$	(diameter of small pipe)
V	$= 6.5 \text{ ft/sec}$	(flow velocity in pipe)
L	$= 3,000 \text{ ft}$	(length of pipe)

The values of the Reynolds numbers for the two cases are:

$$Re_{D_1} = 1.2 \times 10^7,$$

$$Re_{D_2} = 2.3 \times 10^6$$

For the pipes of different diameter, the roughness height is assumed to be the same. The friction factor is then a function of the Reynolds number and the relative roughness ϵ/D (D is the pipe diameter).

The head loss due to friction in the pipes is

$$h_f = \frac{L}{D} \frac{V^2}{2g} f\left(\frac{\epsilon}{D}, Re\right) \quad (3-10)$$

This quantity is the energy loss per unit weight due to friction in the pipe. The pressure loss due solely to the frictional losses along the pipe is then given by

$$\Delta p_f = \rho g h_f$$

The necessary power (which must be factored up by pump efficiency) is obtained by multiplying the pressure loss by the volume flow rate

$$P_f = \rho g h_f VA$$

where A is the pipe cross-sectional area.

These calculated values are given in Table 3.2-5. The total cross-sectional area for the multiple pipe bundle is assumed to be the same as for the 30-ft diameter pipe.

Table 3.2-5. Head Loss, Pressure Loss, and Power Use Due to Friction for Several Values of the Roughness Height

ϵ (ft)	h_{f1} (ft)	h_{f2} (ft)	Δp_{f1} (psi)	Δp_{f2} (psi)	P_{f1} (MW)	P_{f2} (MW)
0.01	1.01	7.4	0.44	3.2	0.39	2.9
0.001	0.69	4.5	0.30	2.0	0.27	1.8
0.0001	0.54	3.6	0.24	1.6	0.21	1.4
0.0	0.50	3.3	0.22	1.4	0.19	1.3

If a number of 6-ft pipes is increased to allow the same volume flow rate with a lower average velocity, then the losses can be reduced. As the velocity in the pipe is decreased, the friction factor increases only slightly. From Equation (3-10) the head loss decreases approximately as the velocity squared. If V_{RED} denotes a reduced velocity, then to affect the same losses for a bundle of 6-ft pipes as for a single 30-ft pipe, the flow velocity must be reduced by

$$\frac{V_{RED}}{V} = \sqrt{\frac{h_{f2}}{h_{f1}}} = 0.39$$

for the smooth pipe. (For the largest roughness height in Table 3.2-5, the ratio V_{RED}/V is 0.37, and the other values lie in between.) To maintain the same flow rate, this reduction in velocity requires that the total cross-sectional area increase by 2.6 times. More than sixty-four 6-ft pipes would be required to keep the losses equal. The velocity V_{RED} for the sixty-four-pipe bundle would be 2.5 ft/sec.

To demonstrate the behavior of the head loss (or necessary power) for bundles of pipes, Figure 3.2-14 shows the dependence of head loss and power use on the number of pipes for 6-ft diameter pipes. The roughness height is indicated on each curve.

In Figures 3.2-15 and 3.2-16 the dependence of the head loss on the average pipe velocity is given for various pipe diameters and a constant volume flow rate of 4585 cubic ft/sec. (This corresponds to 0.65 ft/sec average velocity in a 30-ft diameter pipe.) The increasing velocity for

Figure 3.2-14. Head loss due to friction in a bundle of 6" pipes versus the number of pipes, given for 4 values of the roughness height, ϵ . The total volume flow rate is held constant at 4585 cubic ft/sec. To convert the head loss to the power requirement use,

$$P \text{ (MW)} = h_f \text{ (ft)} \times .3885 \left(\frac{\text{MW}}{\text{ft}} \right)$$

a given pipe diameter is caused by adding more pipes to the bundle. The average velocity is defined as the value flow rate divided by the number of pipes times pipe cross-sectional area. The 30-ft pipe head loss is indicated on these figures. Notice that the necessary power is proportional to the head loss in these figures because the volume flow rate is held constant.

If a minimum average velocity and maximum allowable head loss were imposed as design constraints, they would divide Figures 3.2-14 and 3.2-15 into regions of acceptable and unacceptable values. The velocity of 3 ft/sec and head loss bounds is an example. The hypothetical pipe parameters then designate the curves which pass through the region in the lower right corner of these figures as the acceptable designs. The sensitivity of the OTEC plant to head loss in the pipe is necessary in order to evaluate these designs.

Figure 3.2-15. Head loss due to friction in smooth pipe bundles versus velocity for several pipe diameters.

Figure 3.2-16. Head loss due to friction in bundles of rough pipes (roughness height = 0.01 ft) versus the average velocity for several pipe diameters.

3.3 STRESS ANALYSIS

3.3.1 Approach

The various proposed configurations have been analyzed to determine their capability to sustain the hydrodynamic environment. Calculations were performed for both the extreme and the operational (Sea State 5) environments as specified in the Environmental Package. To determine the loads, the TRW CWP computer model was used with an input of eight superimposed sinusoidal waves whose frequencies uniformly spanned the critical region of the wave spectrum and whose amplitudes were adjusted to simulate the spectrum energy distribution. The program output was the peak values of the axial and bending stresses, the joint rotations, and the lateral force as a function of beam location. The values of the maximum compressive (buckling) pressure acting on the pipe due to the relative motion of the pipe and the water were computed from the lateral force. To compute the total buckling pressure, these values were then added to the values of buckling pressure resulting from the internal flow.

The CWP is subject to several modes of failure, each of which was investigated separately and is discussed below.

3.3.2 Tensile Strength

Due to the combination of the tensile loads resulting from the weight of the pipe and its ballast and the bending moments, the pipe is subjected to a peak tensile load which was compared with the material yield or ultimate strength for infrequent loads or to its fatigue strength for repeated loads. For the CWP, the problem of fatigue strength is particularly severe because of its required life of 30 years without repair and because of the effect of seawater on the fatigue strength.

The CWP is subjected to a complete spectrum of loads ranging from the extreme (hundred-year hurricane) condition to a calm sea. To obtain a complete fatigue analysis, the entire spectrum of load versus frequency of occurrence would have to be analyzed. However, calculations for the extreme condition and the maximum operating condition indicate that the loads for the maximum operating condition are significantly less than for the extreme condition. In addition, the frequency curves from the

environmental package indicate that the number of waves exceeding the ten-foot significant wave height of the maximum operating condition is only 2 percent. For a first approximation, therefore, the maximum stresses resulting from the extreme condition can be compared with the ultimate (or yield) stress of the material, while the maximum stresses for the operational condition can be compared with the fatigue strength for 10 cycles (one cycle every 1.5 min for 30 years). For the final design cycle a more detailed fatigue analysis may be desirable.

3.3.3 Compressive Capability

Under compressive loads the basic material may fail (the compressive strength of fiberglass is typically less than its tensile strength); however, a more likely failure mode for the relatively thin walled ($R/t = 180/2 = 90$) CWP is elastic buckling for which, initially at least, the material yield strength is not exceeded. For materials with a relative low strain capability, such as steel and fiberglass, the resulting deformations may cause the material yield strength to be exceeded and thus result in permanent damage. For materials with high strain capability, such as elastomers or polyethylene, buckling can occur without any permanent damage and is permissible for the non-operational conditions.

Several types of buckling must be considered: The first is the usual short-wave shell buckling mode typical for medium length cylinders subjected to axial loads. The buckling wave lengths in both the axial and circumferential directions are approximately equal to $\sqrt{Rt}$, where R is the cylinder radius and t is the effective cylinder wall thickness. For a steel CWP with a 1-in. wall, the buckling wave length is approximately 13 in. To increase the buckling strength requires either increasing the thickness or adding longitudinal stiffeners spaced no more than 13 in. apart and supported with circumferential stiffeners (rings). Optimization studies have indicated that for cylindrical shells, the use of longitudinal stiffeners alone is inferior to simply increasing the wall thickness. An alternate method, which is particularly appropriate to fiberglass, is to use a double-wall sandwich construction with some kind of continuous core in between. The core provides a uniform support for each of the walls, thereby preventing the local buckling that can occur with discrete stiffeners that are too widely spaced.

A second type of buckling caused by axial bending moments can occur for very long cylinders. It involves the flattening of the cross section, and results from the inward force components caused by the tensile forces in the outer fibers and the compressive forces in the inner fibers of a bent beam. For bending moments less than the critical moment, the flattening can also cause a decrease in the bending stiffness. This phenomenon, sometimes called Brazier buckling, was first investigated by Brazier in 1927 and subsequently by Reissner. Both of these analyses were for infinitely long cylinders and indicated that for an isotropic cylinder, the Brazier buckling stress was of the order of the short-wave buckling stress. Figure 3.3-1 shows the computed values of the short wave and Brazier buckling stresses for a steel CWP as a function of wall thickness. It can be seen that the infinite length Brazier mode is only critical for thicknesses less than 1.4 in., and then only slightly. Later analyses by Yao and Stephens, Starnes and Almroth (Reference 3-54) included the effect of finite lengths and indicate that the addition of stiffening rings significantly increases the capability against this type of buckling. More details of these analyses are given in Appendix C.

Figure 3.3-1. Axial Buckling Stress for Steel Pipe

Buckling due to lateral pressure has similar characteristics to Brazier buckling: the buckling mode forms one-half wave between supports (i.e., rings) and the buckling pressure is proportional to the spacing between supports. However, the number of circumferential waves is a function of the length-to-diameter ratio and the thickness-to-diameter ratio of the cylinder wall. Because the buckling pressure is inversely proportional to the ring spacing, a relatively few rings are very effective in increasing the buckling pressure. The use of even, widely spaced rings has the additional effect that ovaling caused by the non-uniform hydrodynamic pressures is significantly reduced. Figure 3.3-2 shows the lateral buckling pressures of a steel CWP as a function of wall thickness for various values of ring spacing, where the ring is assumed large enough to provide simple, effective support. The major increase in buckling pressure due to even fairly large values of ring spacing is quite evident.

Figure 3.3-2. Lateral Buckling Pressure for Steel CWP

While considerable analytic work has been done on the buckling of cylinders subjected to uniform external pressure, relatively little work has been done on cylinders subjected to pressure varying in the circumferential direction, such as the CWP is subjected to by the action of the inertia and pressure forces of the water. Only two papers were found (see Appendix C) and their basic conclusion is that because of the local nature of the peak pressure distribution, the allowable peak pressure is increased. The amount of the increase is a function of the relative amplitudes of the pressure peak and the uniform pressure on which it is superposed. The value of the increase (which depends on the L/R and R/t ratios of the cylinder) increases from zero to 40 percent as the ratio of the non-uniform pressure to the maximum pressure increases from zero to 1.0. For the CWP, the non-uniform pressure due to the hydrodynamic forces is typically less than 50 percent of the maximum pressure differences; thus, the increase in the allowable pressure is about 25 percent. More details are given in Appendix C.

The individual items contributing to the lateral buckling pressure on the pipe are listed in Table 3.3-1 for both sites in both the operational and the non-operational extreme conditions. For the non-operational condition it is assumed that there is no water flow in the pipe, but that the density gradient still remains. All of the pressure components cause a uniform lateral except for that due to the lateral hydrodynamic forces. The pressure components are summed to obtain the maximum expected values. These values are then multiplied by the appropriate factor of safety to obtain the ultimate value. The maximum ultimate pressure of 7.4 psi occurs at the top of the Puerto Rico pipe in the operating condition. The next highest value is 4.7 psi for the Brazil site operational condition.

To account for the uncertainties in the applied loads and the pipe strength capabilities, a safety factor of 2.0 is used for the operational condition and a factor of 1.5 for the extreme condition. The lower factor is used for the extreme condition because of its unlikely occurrence. This factor is primarily used to determine the required buckling capability; for the tensile capability, fatigue is usually critical and for that condition the lower bound of the scatter curve is used for the required life.

Table 3.3-1. CWP Lateral Pressure Requirements

 ΔP (PSI), EXTERNAL - INTERNAL

● BRAZIL SITE

CAUSE	OPERATING		NON-OPERATING	
	BOTTOM	TOP	BOTTOM	TOP
ENTRY LOSS	0.34	0.34	0.0	0.0
FLOW VELOCITY	0.29	0.29	0.0	0.0
FRICTION	0.0	0.25	0.0	0.0
TEMPERATURE DIFFERENCE	0.0	0.87	0.0	0.87
PRESSURE DUE TO LATERAL FORCE	0.1 - 0.4	0.6	0.1 - 1.2	0.6 - 1.2
TOTAL	0.7 - 1.0	2.4	0.1 - 1.2	1.5 - 2.1
ULTIMATE PRESSURE	1.5 - 2.0	4.7	0.2 - 2.4	2.2 - 3.1

● PUERTO RICO SITE

CAUSE	OPERATING		NON-OPERATING	
	BOTTOM	TOP	BOTTOM	TOP
ENTRY LOSS	0.34	0.34	0.0	0.0
FLOW VELOCITY	0.29	0.29	0.0	0.0
FRICTION	0.0	0.30	0.0	0.0
TEMPERATURE DIFFERENCE	0.0	2.05	0.0	2.05
PRESSURE DUE TO LATERAL FORCE	0.02 - 0.25	0.7	0.3 - 2.0	0.5 - 1.5
TOTAL	0.65 - 0.88	3.7	0.3 - 2.0	2.5 - 3.6
ULTIMATE PRESSURE	1.3 - 1.8	7.4	0.5 - 3.0	3.8 - 5.3

3.3.4 Results3.3.4.1 Steel Designs

Three types of steel designs were investigated: the first was the monolithic design with no movable joints which appears to be the simplest configuration because of the absence of moving parts. Because of the stiffness of steel, however, stresses in the pipe are quite high and require high-strength steel, high-quality welds, and high-strength and heavy joints.

To avoid this problem, the second configuration type assumed the use of a large number of "simple" joints which joined the two adjacent sections at their outer circumference. The difficulty with this type of joint is that the large radius of the pipe limits this type of joint to very small angles of rotation.

The third approach was to use a relatively small number of joints with a large angular capability. This configuration requires a gimbal-type joint in which the load is transferred to bearings or a ball. Because there would be only a few of them, however, they could be more expensive on an individual basis.

a. Monolithic. The results of the analysis for the monolithic configuration (no moving parts) for the extreme conditions at the Brazil and Puerto Rico sites are shown in Figure 3.3-3. The computed maximum tensile and compressive stresses, multiplied by the extreme condition safety factor of 1.5, are plotted as a function of the pipe wall thickness. The variation in the axial buckling strength of a monocoque cylinder is also shown.

Figure 3.3-3. Computer Results for Monolithic Steel Pipe

It can be seen that the computed stresses vary slowly with thickness, with a slight minimum at $t = 1.5$. Increased thickness (and therefore stiffness) tends to increase the dynamic response due to the wave motion, while response to the surface current becomes predominant for the thinner sections.

The buckling strength equals the compression load at a thickness of about 1.5 in. (slightly higher for Puerto Rico), so an unstiffened monocoque design must have a thickness of at least 1.5 in.

Thicknesses less than 1.5 in. require stiffening. However, stiffening to increase the axial buckling capability requires relatively closely spaced stiffeners in both the axial and circumferential direction, so the stiffening system will be quite complex and expensive. Our primary emphasis for the steel monolithic configuration, therefore, was on the monocoque design, using widely spaced ring stiffeners only to obtain the required lateral pressure capability.

Figure 3.3-4. Envelope of Maximum and Minimum Stress Rigid Steel, Fixed Joints (1.5-in. Thick) Brazil - Barge

Figure 3.3-4 compares the maximum bending steel distributions for a 1.5-in. wall thickness pipe subjected to the Brazil extreme and operational conditions. In this plot (and all subsequent, similar ones), a positive

bending stress indicates the absolute value of the bending stress resulting from a positive bending curvature and vice versa. Thus, the figure shows the bending stresses that occur at the positive and negative limits of the dynamic oscillation. The two peaks in the distribution indicate that the pipe is oscillating primarily in its second bending mode.

Although the maximum bending stress for the extreme condition is 46 ksi, the maximum bending stress for the operational condition is only 4 ksi. This value of oscillating stress is well within the basic material fatigue strength of HY80 steel for 10 million cycles. The critical items will be the circumferential assembly joints, however, which will have to be designed for a tensile load intensity of $80,000 \text{ psi} \times 1.5 \text{ in.} = 120,000 \text{ lb/in.}$

b. Multi-Jointed. The second series of steel configurations studied was multi-jointed designs that used a large number of simple joints with a relatively small angular capability. Conceptually, these would be joints in which external flanges separated by some flexible material (elastomer) would be clamped together, with the deformation of the material allowing for some relative rotation. These would be similar to many flexible joints used on smaller pipes and have the advantage that the loads are distributed around the circumference and there are no highly loaded rubbing surfaces or bearings. The size of the CWP places a severe limit on the angular motion of such joints, however, and if θ is the rotational angle of the joint measured in degrees, and R is the pipe radius in inches, then the axial displacement at the pipe wall is

$$\Delta = \frac{R\theta}{57.3} = 3.14 \theta \text{ in.}$$

To limit the axial strain to 0.10 (10 percent), the original height of the compressible material would have to be $10 \times 3.14 = 31 \text{ in.}$ per degree of permissible rotation. Another possibility is to take advantage of the higher shear strain capability of elastomers and use a spherical type such as ABAM proposed for their concrete design. Some of the gain in shear strain capability is lost because the radius, R , used in the equation for shear strain becomes the increased radius of the spherical joint, rather than the radius of the pipe. In any case, to keep the

complexity and cost of the joint within bounds, it is desirable to keep the angular motion of the joint below one degree.

A series of calculations were made with variable joint stiffnesses for a 1-in. thick steel pipe with joints every 100 feet and subjected to the Brazil extreme spectrum. The results are shown in Table 3.3-2: the peak joint angles and the peak values of the maximum and minimum axial stresses (negative values indicate compression) and the maximum dynamic pressure due to the relative motion of the water and the pipe are listed for each segment beginning from the top of the pipe. The three joint stiffnesses analyzed were 3.2E8, 3.5E9, and 1.9E10, giving equivalent elastic moduli of the pipe (assuming the 100-ft sections of the pipe were rigid) of 0.25E6, 2.5E6, and 10.E6 psi. Thus the stiffness of the pipe with the most flexible joint was 1/120 that of a pipe without joints. For the most flexible joint, the maximum joint angle is 1.6 deg, decreasing to 0.8 deg as the joint stiffness is increased by a factor of 40. Although the angle requirement is cut in half, it is still quite large to be achieved with a simple joint.

Table 3.3-2. Results for Multiple Jointed Steel Pipe

JOINT STIFFNESS (FT LB/RAD)	3.2E8	3.5E9	1.9E10	INF
EQUIVALENT E (PSI)	0.25E6	2.5E6	10.0E6	30.0E6
PEAK JOINT ROTATIONS (DEG)	1.4	1.1	0.7	
	0.9	0.9	0.7	
	0.9	0.9	0.8	
	1.1	1.2		
	1.6			
MAXIMUM TENSILE STRESS (KSI)	8.8	14.8	24.4	52.5
	4.5	11.6	22.3	50.0
	3.5	9.3	23.6	
	2.6	17.4		
	1.9			
MINIMUM TENSILE STRESS* (KSI)	4.3	-2.2	-14.0	-41.2
	2.1	-2.0	-15.8	-45.8
	1.3	-3.9	-21.1	
	0.5	-7.1		
	-0.6			
MAXIMUM PRESSURE (PSI)	0.72	0.73	0.66	0.72
	0.48	0.54	0.63	0.73
	0.18	0.41	0.54	0.71
	0.19	0.43	0.60	1.18
	0.19	0.45	1.06	
	0.47	0.82		

*NEGATIVE VALUES INDICATE COMPRESSION.

VALUES ARE LOCAL PEAKS STARTING FROM THE TOP. PEAKS OCCUR AT ANTI-NODES OF VIBRATION MODES.

The tensile and compressive stresses increase with the joint stiffness, as expected, but the values for the stiffest joint (1.9×10^{10}) indicate that a design with a 40,000 psi yield strength steel and a wall thickness of 0.75 inches is feasible. Any further increase in the joint stiffness in an attempt to further decrease the joint angle would cause a rapid increase in the wall stresses, and would necessitate a both higher strength steel and a thicker wall.

It should be noted that the hydrodynamic pressures at the top of the pipe are essentially independent of the joint stiffness since they are presumably controlled by the motion of the barge. However, the pressure at the bottom of the pipe increases from a relatively low value 0.4 psi for the most flexible joints, to the maximum value of 1.18 psi for the rigid joint configuration. This variation with pipe stiffness is typical of most of the computer results.

c. Gimbal Joints. An alternate approach to the flexible joint design is to use a more complex joint with a larger rotational capability and to minimize the number of joints. A joint with this larger angular capability would require some kind of gimbal or universal joint construction using bearings.

A series of runs were made with 2, 3, 4, and 5 joints, both uniformly spaced and with the joints concentrated near the top where the most motion was expected to occur. The results are shown in Table 3.3-3; the maximum rotation is shown for each joint (joint 0 is the ship connection), and the maximum and minimum values of the axial stress and the maximum hydrodynamic pressure for each pipe segment. It has been suggested that one or two joints would eliminate the bending stress problem, but these results indicate that at least 3 joints would be required. With 3 joints, the maximum compressive stress is 7.3 ksi; with 4 or more joints all compressive stresses are essentially eliminated. Buckling, therefore, is no longer a consideration except during handling and assembly. To minimize handling difficulties, a 0.75-in. minimum thickness was assumed so that the buckling strength was equal to the tensile-yield strength (40 ksi) of the low-strength steel. Calculations were then performed for a 0.75-in. thick pipe with 5 joints for the extreme and operational Brazil spectrum. The results, given in

Table 3.3-3. Results for Steel Gimbal Joint Designs, Brazil Extreme Conditions

NO. OF JOINTS SPACING (FT) WALL THICKNESS (IN.)		2 833/UNIF 1.0	3 625/UNIF 1.0	3 400 1.0	4 500/UNIF 1.0	4 400 1.0	5 417/UNIF 1.0	5 417/UNIF 0.75
MAXIMUM ANGLES (DEG)	0	14.7	15.3	23.1	15.8	19.8	18.1	17.1
	1	3.64	3.91	4.1	4.6	4.6	5.1	5.2
	2	1.22	1.27	3.7	2.1	2.9	2.9	2.5
	3		0.28	12.2	0.9	3.2	1.7	1.2
	4				0.6	7.7	1.0	0.6
	5						0.7	0.7
MAXIMUM TENSILE STRESS (KSI)	1	46.4	22.2	11.3	15.7	11.7	12.3	13.8
	2	26.4	12.9	9.2	13.0	9.8	10.1	10.1
	3	10.9	6.6	8.7	6.8	7.4	6.8	6.4
	4		2.8	17.8	4.3	5.8	4.9	4.8
	5				2.2	3.4	3.0	2.9
	6						1.0	1.0
MINIMUM TENSILE STRESS (KSI)	1	-34.0	-7.3	2.4	-0.8	2.0	1.3	-0.3
	2	-18.9	-1.7	2.2	0.7	2.3	1.7	1.1
	3	-7.7	0.9	0.3	2.2	1.6	2.5	2.3
	4		1.0	-1.4	1.7	0.8	1.6	2.1
	5				0.9	0.7	1.4	1.5
	6						0.3	0.3
MAXIMUM PRESSURE (PSI)	1	1.1	0.7	0.7	0.7	0.7	0.7	0.7
	2	0.6	0.5	0.8	0.6	0.7	0.7	0.6
	3	0.4	0.2	0.3	0.2	0.3	0.3	0.2
	4		0.1	0.5	0.1	0.2	0.2	0.1
	5				0.03	0.1	0.1	0.04
	6						0.04	0.02

Table 3.3-4, show the expected rapid decrease in joint rotations and joint load with depth, and the major reductions in the joint rotations and the variation in the joint force ($F_{MAX} - F_{MIN}$) between the extreme and the operational conditions.

Table 3.3-4. Joint Angles and Loads for Steel Pipe with 5 Uniformly Spaced Gimbal Joints — Brazil Spectrum

JOINT	DEPTH	EXTREME CONDITION			OPERATIONAL CONDITION		
		θ_{MAX} (DEG)	F_{MAX} (10^6 LB)	F_{MIN} (10^6 LB)	θ_{MAX} (DEG)	F_{MAX} (10^6 LB)	F_{MIN} (10^6 LB)
0	60	18.1	7.6	5.04	2.82	6.35	6.30
1	476	5.22	6.3	4.21	0.132	5.30	5.26
2	892	2.46	5.1	3.37	0.087	4.25	4.22
3	1308	1.23	3.8	2.54	0.114	3.20	3.18
4	1724	0.61	2.6	1.70	0.228	2.16	2.14
5	2140	0.71	1.3	0.87	1.15	1.11	1.10

Under operational conditions, the joints sustain an essentially constant load. Note, however, that the rotation of the bottom joint is actually larger for the operational condition than for the extreme condition.

These results indicate that a 3-jointed design with 0.75-in. thick, 40 ksi steel is practical.

3.3.4.2 Fiberglass Designs

a. Sandwich. The initial parametrical numerical results for fiberglass were obtained with slightly different hydrodynamic coefficients than the current values, and by assuming a 0 to 90 deg ply angle layup. Although qualitatively valid, these results may differ numerically from the latest values; they are presented to show the approach used in selecting the proposed fiberglass design. The results for the Brazil extreme spectrum are shown in Table 3.3-5. They indicate that the tensile stress is satisfactory, but to obtain an adequate buckling capability, including a 1.5 factor of safety, thicknesses greater than 2 inches are required. To avoid increasing the thickness beyond 2.0, the approach of using a sandwich design was investigated. In a sandwich design the original thickness of material is split in half and separated by a lightweight core. The core serves to transmit normal and shear forces between the two forces, but by itself contributes nothing to the axial or bending stiffness. By separating the two face sheets, however, the bending stiffnesses are materially increased, thus increasing the compression buckling capability. The function of the core is to continually support the two face sheets so that they act together and do not buckle individually. The buckling strength of the wall can then be increased without any increase in the structural weight.

Table 3.3-5. Initial Parametric Results for FRP, Brazil Extreme Spectrum and Current

WALL THICKNESS (IN.)	MAXIMUM TENSILE STRESS (PSI)	MAXIMUM COMPRESSIVE STRESS (PSI)
1	11,570	-10,100
1.5	8,500	-7,400
2.0	8,200	-7,064

$E = 3 \times 10^6$ PSI.

THESE VALUES SHALL BE COMPARED TO AN ESTIMATED 10^7 CYCLE FATIGUE CAPABILITY OF ABOUT 10,000 PSI.

The use of a fiberglass sandwich is particularly advantageous for the CWP. With a lightweight foam or balsa wood core, the density of the pipe can be adjusted so that it is near neutrally buoyant, can be towed out to the site, and erected without exceeding its strength capability. It can thus be assembled on shore, rather than at sea. A second advantage is that the stress in the pipe wall due to the wave motion is essentially independent of the wall thickness. Thus, doubling the wall thickness will double the loads which will have to be transmitted by the assembly joints. This means that the joints will have to be thicker to maintain the same axial stress and longer to maintain the same critical shear stress between the wall and the joint.

The results of the parametric study are shown in Figures 3.3-5, -6, and -7. Figure 3.3-5 compares the maximum bending stress (BST), including the 1.5 factor of safety, with the axial buckling stress as a function of the core thickness for a wall thickness of 1.0 in. The required core thickness is 2.1 in. to prevent buckling, which is significantly greater than the thickness required for neutral buoyancy.

Figure 3.3-5. Buckling Result for Fiberglass Sandwich Cylinder T = 1.0

Figure 3.3-6 shows the same results for a wall thickness of 1.5 in. In this case, the buckling stress is shown for equal longitudinal (0 deg) and circumferential windings ($t_c/t_a = 1$) and for a reduced ratio of circumferential windings ($t_c/t_a = 0.67$). It is clear that an equal value is preferable.

Figure 3.3-6. Buckling Result for Fiberglass Sandwich Cylinder T = 1.5

The core thickness required for buckling strength is 0.9 in., while that required for neutral buoyancy is 1.8 in. A core designed for neutral buoyancy will therefore have a significant buckling margin.

Figure 3.3-6 also shows the lateral buckling pressures P_{CR} for an infinite length cylinder ($L = \infty$) and for a cylinder with rings stiff enough to provide simple support spaced every 100 ft (it was initially assumed that each fabricated cylinder section would be 100 ft long). The major increase in buckling pressure due to the ring is evident. A discussion of the calculations to determine the ring size is given in Appendix C.

Figure 3.3-7 shows the same results for a 2-in. wall thickness. Here the core thickness required by buckling is 1.1 in., while that required for neutral buoyancy is 2.5 in. A 2-in. wall thickness sandwich is therefore considerably oversized.

Figure 3.3-7. Buckling Result for Fiberglass Sandwich Cylinder T=2.0

Figure 3.3-8 shows the distribution of bending stresses for a 1.5-in. fiberglass pipe with a 2-in. foam core subjected to the Brazil extreme and operational conditions. The significant reduction in going from the extreme to the operational conditions is evident. Table 3.3-6 compares the maximum tensile, maximum compressive stress, and maximum dynamic pressure for the Brazil and Puerto Rico extreme conditions and the Brazil operational condition with a uniform 1-knot current. The values shown are the local peaks starting from the top, and are usually associated with the antinodes of the predominantly excited vibration mode. The values for the maximum stresses for both sites are essentially the same,

Figure 3.3-8. Envelope of Maximum and Minimum Bending Stresses
FRP Sandwich Fixed Joints Brazil - Barge

but the Puerto Rico pressures are slightly higher. The maximum amplitude of the oscillating stresses (Figure 3.3-8) is 6.5 ksi for the extreme condition and 2 ksi for the operating condition. Both of these values are considerably less than the estimated 10 ksi fatigue stress for fiberglass at 10 million cycles and indicate that fatigue should not be a problem.

The buckling results shown in Figures 3.3-5, -6, and -7 were all based on a 0 to 90 deg ply layup on the assumption that this was the simplest layup that gave high axial strength. Subsequent analyses, however, have indicated that a 50 percent 0 deg, 50 percent ± 60 deg layup gives significantly higher buckling strengths (due to the increased shear stiffness) and at least equal axial strength (due to the axial component of the 60 deg plies). The lateral buckling pressure is slightly lower, but this can be compensated for by adjusting the ring spacing.

Table 3.3-6. Results for 1.5-inch Fiberglass Design with 2-inch Core

CATEGORY	CONDITION*		
	BRAZIL EXTREME	PUERTO RICO EXTREME	OPERATIONAL 1 KNOT
MAXIMUM TENSILE STRESS (KSI)	13.1	13.0	3.2
	11.2	10.8	4.0
	9.0	11.0	4.3
	8.1	13.4	4.2
			4.0
			3.5
MINIMUM TENSILE STRESS (KSI)	-10.9	-11.1	-1.0
	-8.9	-9.5	-1.7
	-6.7	-10.1	-2.0
	-5.7	-13.0	-2.0
			1.7
			-1.1
MAXIMUM PRESSURE (PSI)	0.70	1.27	0.63
	0.51	0.84	0.20
	0.40	0.86	0.20
	0.45	0.80	0.19
	0.39	1.02	0.19
	0.33	1.06	0.20
		1.84	0.24

*VALUES ARE LOCAL PEAKS STARTING FROM THE TOP.

It then appears feasible to reduce the 1.5-in. wall thickness to perhaps 1.25-in. with an accompanying reduction in the core thickness.

b. Pultrusion. The pultrusion concept consists of longitudinal hollow tube pultrusions that are mechanically snapped together, then bonded and enclosed within inner and outer specially wrapped pultrusions. This is basically similar to the sandwich design with 0 to 90 deg. plies, and the dimensions would be about the same.

c. Solid Fiberglass. For the sandwich configuration, the buckling capability of the dozen layup was not critical since it was fairly simple to compensate for by slightly increasing the core thickness. For a solid configuration, however, this could only be compensated for by adding more glass. A study was therefore made to determine the best layup configuration and the results are listed in Table 3.3-7. Initial

results indicated that shear stiffness was very important and that a configuration using 0, ± 60 deg plies had a much higher buckling load than an equivalent 0, 90 deg orientation. It was also found that a large number of ply layers was required to attain an approximately equal ratio of bending stiffness in the axial and circumferential direction. Eight layers were taken as a compromise value; more layers would give somewhat higher buckling loads, but would be more difficult to fabricate.

Table 3.3-7. Layup Configuration Analysis Results

THICKNESS OF 60° PLIES (IN.)	THICKNESS OF 0° PLIES (IN.)	EFFECTIVE AXIAL MODULUS (PSI X 10 ⁻⁶)		EFFECTIVE BENDING MODULUS (PSI X 10 ⁻⁶)		AXIAL BUCKLING STRESS σ_{CR} (KSI)
		E_X	E_Y	E_{BX}	E_{BY}	
0.250	0.375	4.0	1.78	3.03	2.28	10.9
0.3125	0.3125	3.36	2.10	2.53	2.71	11.3
0.365	0.260	3.06	2.38	2.16	2.95	11.1
0.4167	0.2083	2.64	2.64	1.84	3.15	10.6

The effective axial module E_X and E_Y are obtained by dividing the computed stiffnesses by the thickness; the effective bending stiffnesses are obtained by dividing the computed bending stiffnesses by $t^3/12$. It can be seen that the highest buckling stress, 10.9 ksi, is obtained for the case of equal ply thicknesses in the 0 and ± 60 deg direction. For this case, the bending moduli in the axial and circumferential direction are nearly equal, which indicates that this is a major influence on the buckling load.

The computed stresses for a 2.75-in. solid fiberglass pipe are shown in Figure 3.3-9 for the extreme and operational Puerto Rico conditions. Table 3.3-8 shows the maximum tensile, maximum compression, and maximum hydrodynamic pressures for extreme and operational conditions at both sites. The maximum compression stresses for Puerto Rico are slightly higher.

The conclusion is that a 2.5-in. thick pipe would be satisfactory for the Brazil site, but that the Puerto Rico site would require an increase to 2.75-in.

Figure 3.3-9. 2.75-inch Per Cold Water Pipe, Puerto Rico

Table 3.3-8. Results for 2.75 Solid Fiberglass Design

	CONDITION*			
	BRAZIL EXTREME	PUERTO RICO EXTREME	BRAZIL OPERATIONAL	PUERTO RICO OPERATIONAL
MAXIMUM TENSILE STRESS (KSI)	8.5 6.5 6.7	8.8 7.3 7.8 9.5	1.8 1.7 1.6 1.2 1.3	2.3 2.1 1.5 1.3 1.1
MINIMUM TENSILE STRESS (KSI)	-7.1 -5.6 -6.3	-7.2 -5.9 -7.1 -9.2	-0.3 -0.6 -0.7 -0.9 -1.1	-0.6 -0.7 -0.6 -0.7 -0.8
MAXIMUM PRESSURE (PSI)	0.69 0.63 0.48 0.55 0.96	1.09 0.82 0.99 1.01 1.83	0.62 0.21 0.20 0.20 0.38	0.23 0.16 0.14 0.15 0.27

*VALUES ARE LOCAL PEAKS STARTING FROM THE TOP

3.3.4.3 Concrete Designs

Prestressed and reinforced concrete is a candidate for the OTEC CWP, and its advantages and disadvantages are presented in Table 3.3-9. The low materials cost, the ability to tailor reinforcement (prestress) to the loads, and the ability to mold into complex shapes (e.g., the joint areas of the articulated CWP concepts) are all advantages that will yield a low-cost CWP configuration.

Table 3.3-9. Concrete CWP Material

ADVANTAGES	DISADVANTAGES
LOW COST OF MATERIALS: \$800/CUBIC YARD VERSUS \$3240/TON STEEL	LOW TENSILE RESISTANCE IN DIRECT TENSION, FLEXURE, AND DIAGONAL TENSION (SHEAR), ($f_T = 0.1 f'_c$)
WIDE MATERIAL AND SEMI-SKILLED LABOR AVAILABILITY RESULT IN LOW COSTS AND IMPROVED CONSTRUCTION-DEPLOYMENT SCHEDULES	HIGH-WEIGHT TO STRENGTH RATIO
ABILITY TO TAILOR REINFORCEMENT (PRESTRESS) TO LOAD	LIMITED DUCTILITY ($\mu \leq 2.0$)
ABILITY TO MOLD INTO COMPLEX SHAPES AND DOUBLE CURVATURE CONFIGURATIONS LEADING TO OPTIMUM STRUCTURAL DESIGNS	HANDLING HEAVY SECTIONS DURING DEPLOYMENT
DURABILITY AND REDUCED MAINTENANCE	SPECIAL PREPARATION NEEDED TO ENSURE WATER TIGHTNESS AT JOINT
FATIGUE RESISTANCE	
GOOD THERMAL INSULATING QUALITIES (THERMAL CONDUCTIVITY ONE-SIXTH THAT OF STEEL)	

Concrete's low tensile strength and high weight-to-strength ratio are significant disadvantages that the designer must overcome during the analysis of the CWP cross sections. CWP design concepts that are supported by the platform with a gimbal attachment (with tensile loads taken through the walls) are dominated by the dynamic axial tensile and bending loads. Concrete, by itself, has a relatively low tensile strength: about 10 to 15 percent of its compression strength. Concrete members subjected to tensile stresses higher than these values may develop hairline cracks

that could lead to corrosion of the rebars and prestressing tendons (see subsection 3.1.5.1).

Structural lightweight concrete (less than 125 pcf) attractive to the CWP concept evaluation because of low cost and weight and associated with minimum wall thicknesses, is of prime importance for this pipe configuration. The minimum thicknesses are determined by the strength requirement for the loads, as well as by practical considerations for cover and placement of reinforcing steel and prestressing ducts. Previous TRW, IR&D, and DOE-funded studies have shown that standard weight (greater than 125 pcf), high-strength concretes would yield costly CWP configurations and result in platform loads greater than that of a steel configuration. The use of high-strength, small size, coarse aggregates and high-cement contents, with the possible use of water-reducing admixtures, can produce lightweight concrete with strengths comparable to those of standard weight concrete (see subsection 3.1.5).

The design and analysis of the concrete CWP sections were based on the service load analysis (elastic design) method. The CWP concepts were portioned to resist all loads encountered during the required 30-year service life. Reinforcement and prestressing design of the concrete sections were based on the American Concrete Institute (ACI) Building Code Requirements for reinforced concrete, ACI-318-77, and related supplements. The basic material properties of the two concretes and prestressing reinforcing steels are presented in Table 3.3-10. The structural steel used for the center supported universal jointed concrete configuration is also presented. The code requires that the extreme fiber stress in compression be less than 45 percent of the specified compressive strength of the concrete, and that the extreme fiber stress in tension be less than six times the square root of the compressive strength for the design loads. Since the CWP would be submerged for its entire service life, however, and since extensive amounts of concrete cracking could lead to catastrophic failures, the CWP wall sections were portioned with no resultant membrane tension stresses. The CWP configurations were sized so that the resultant stresses on its cross-section during the extreme environmental condition were compression (i.e., zero resultant tension stress). This limitation governed the design of the CWP sections and these assumptions should result in a safe structural design.

Table 3.3-10. Concrete CWP Material Properties

CONCRETE	DRY DENSITY (PCF)	COMPRESSIVE STRENGTH (PSI)	RUPTURE MODULUS (PSI)	POISSON RATIO	YOUNG'S MODULUS (PSI)
LIGHT-WEIGHT CONCRETE	85	4000 TO 4500	500	0.2	1 TO 1.2 X 10 ⁶
MEDIUM-WEIGHT CONCRETE	100	5500	550	0.2	2.4 X 10 ⁶

- PRESTRESSING WIRE
 - ASTM A416 – GRADE 270
 - YOUNG'S MODULUS – 28 X 10⁶ PSI
 - ALL TENDONS SOLID GROUTED WITHIN SEMI-RIGID METAL DUCTS
- REINFORCING STEEL
 - ASTM A615 – GRADE 40
 - YOUNG'S MODULUS – 29 X 10⁶ PSI
- STRUCTURAL STEEL
 - SEE STRUCTURAL STEEL CWP CONCEPTS FOR HY80 STEEL

The CWP configurations were analyzed to determine the buckling strength of the section. The analysis procedures for determination of design loads and buckling strengths were similar to those presented in subsection 3.3 for steel and fiberglass. All concrete CWP configurations will remain stable when subjected to the design loads (external pressure, flexural bending, and axial compressive stresses). The low modulus of elasticity and minimum wall thickness of the lightweight concrete require stiffener rings at 50-ft centers to provide an adequate safety factor for the loads. The stiffener rings were those required for construction of the articulated joint configurations or assembly details.

The service load requirements for the concrete CWP configurations were the combination of peak dynamic axial and bending stresses determined by analysis using the TRW computer model for the extreme environmental sea states (hurricane sea states and currents). The concrete CWP sections were proportioned for these loads. The CWP concepts were analyzed for the extreme and operational (defined as sea state 5 for the analysis)

conditions for the spar platform at the Puerto Rico site and the barge platform at the Brazil site. The sea states and currents for the two sites were specified in the environmental package supplied by NOAA/DOC (see Reference 3-32).

The concrete CWP will see approximately 10^8 waves in its lifetime; fewer than 10 percent will create significant changes in stress. The major portion of the cyclic loadings imposed on the CWP result in stresses much less than high-stress ranges caused by the extreme environmental conditions. The environmental package indicates that the significant wave heights at Brazil and Puerto Rico are less than 12 ft (equivalent to sea state 5) 98 percent of the time. The analysis of the CWP configurations for sea state 5 environmental conditions indicated that CWP stresses were much lower than the extreme hurricane sea states.

Concrete itself shows an endurance limit of about 60 percent in compression. Cyclic loading below this level may actually enhance the fatigue strength of the concrete, especially when the concrete experiences periods of rest (i.e., low stresses) between high ranges of cyclic loading, and when the applied stresses are variable.

The allowable design stresses for prestressed and reinforced members are much less than for steel or fiberglass; high-stress fatigue failure of concrete members is not usually a design concern. As long as the concrete has not cracked, there is little possibility of fatigue failure in steel, both reinforced and prestressing. After concrete cracking, high-stress concentrations exist in the wires at the cracks and may result in partial breakage of the bond between the steel and concrete near the crack. Under repeated loading, the bond may either be completely broken or the steel may be ruptured (Reference 3-55).

The bending moment producing the first hairline cracks in a prestressed concrete beam is computed by the elastic theory, assuming that cracking begins when the tensile stress in the extreme fiber of concrete reaches its modulus of rupture. Test data indicate that the elastic theory is sufficiently accurate up to the point of cracking, and this method is currently used by most design engineers for prediction of cracking load in prestressed concrete beams.

The modulus of rupture is the maximum bending tensile stress that the concrete CWP can withstand without developing hairline cracks. The stresses are calculated assuming that the pipe remains elastic until rupture occurs. The modulus of rupture is a measure of the beginning of hairline cracks which are often invisible to the eye; tensile stress higher than the modulus is necessary to produce visible cracks.

Cracking of the concrete CWP must be limited (i.e., no resultant tensile stress) because it could impair the pipe's durability. Submerged concrete is very susceptible to crack development. Seawater combined with cracked concrete will have an adverse effect on the CWP's service life. Hydrostatic pressures cause a wedging action that accelerates crack growth and leads to steel corrosion and fatigue and possible CWP failure. Furthermore, where crack widths are kept small (at the reinforcement), fully satisfactory endurance life of 10^7 cycles of stress are possible (Reference 3-56).

Cracking can be controlled by properly prestressing the CWP, which allows the engineer to tailor the section reinforcement to create a favorable state of stress for the design loads. Prestressing the concrete sections can ensure that they will remain free of serious structural cracking when subjected to the cyclic loadings of the wave and current environmental loads. Prestressing will help prevent the undesirable fatigue failures associated with concrete members that have developed surface cracks.

Anchorage of post-tensioning tendons require special consideration and should be proven by acceptance tests. Most anchorages systems proposed by supplies have been proven by test to be acceptable for load transfer. Grouting ducts, which transfers stress reversals by bond, will minimize the effects of load cycling on the anchorages. Flexible epoxy coatings on the surface of the concrete may be required to provide additional protection to reinforcement in highly stressed corrosive zones. These requirements for the concrete CWP concepts will be investigated in greater detail during the preliminary design.

Initial prestress in steel minus the losses is known as the effective, or design, prestress. The design prestress is used to determine the required amount of prestressing steel and concrete thicknesses. The losses of prestress result from the following factors:

- Elastic shortening (concrete)
- Creep (concrete)
- Shrinkage (concrete)
- Creep (steel stresses)
- Functional losses.

It is difficult to generalize the amount of prestress loss because it is dependent on many factors: the properties of concrete and steel, curing and moisture conditions, magnitude and time of application of prestress, and the process of prestressing. For average steel and concrete properties, and amount of losses varies from 15 to 20 percent of the initial prestress for post-tensioned members. The amount of losses for the lightweight concrete CWP could be greater than these values. A frequent objection to lightweight concrete for prestressing is its low modulus of elasticity, which indicates more elastic shortening under the same unit stress. This means that there is a slightly higher loss of prestress in the reinforcing steel, and greater elastic movements will result under the application of prestress. The total elastic shrinkage of the lightweight concrete sections was estimated as 0.7 in. The total amount of prestress loss was estimated as 20 percent of the initial prestress for the design analysis of the concrete CWP concepts. A detailed analysis of the prestress losses will be required if a concrete CWP is selected as a concept for preliminary design.

Two generic concrete types of concrete configurations were studied for the CWP concept evaluation: rigid wall and articulated wall joints. The articulated wall joint concepts were further broken-down to a rotational-stiffened and a center-supported universal joint configurations. Of the generic configurations only the articulated wall joint concepts provided satisfactory designs. These configurations were costed and included in the TRW concept evaluation matrix.

The following sections present the specific findings of the concrete CWP configurations evaluated. The general conclusions for concrete CWP pipe configurations is in Table 3.3-11. Concrete appears feasible, but will require several complex articulated joints and a gimbal platform hull attachment.

Table 3.3-11. Concrete CWP Configuration Study Findings

CONCRETE CWP CONFIGURATIONS

- APPEAR FEASIBLE, BUT REQUIRE SEVERAL ARTICULATED JOINTS WHICH REQUIRE DETAILED ANALYSIS, INCLUDING ELASTOMERIC PADS AND STRESS ANALYSIS
- REQUIRE A GIMBAL PLATFORM HULL ATTACHMENT
- REQUIRE LARGE PRESTRESS
- REQUIRE NOMINAL WALL THICKNESSES FROM 6 TO 10 IN.
- REQUIRE SIX MONTHS FOR FABRICATION OF UNITS

a. Rigid Wall. The lightweight concrete material was analyzed as a rigid wall (no articulated joints) CWP concept. The minimum CWP sectional wall thickness determined from the buckling strength requirements established the geometrical properties used as the input for the TRW computer model. It was felt that the lightweight concrete properties (low modulus of elasticity, relatively low wet weight, and minimum wall thicknesses) could survive the extreme environmental sea states. Since concrete is not a very strong material for resisting high localized stresses resulting from articulated joint details (thus requiring large amounts of concrete), and since concrete is a relatively low-cost construction material, this configuration appeared attractive.

The envelope of maximum and minimum bending stresses for the extreme wave spectrum and currents at Brazil is presented in Figure 3.3-10. The detailed stress analysis of the CWP cross section indicated that both the resultant tensile and compressive stresses would exceed the design allowables. Calculated resultant tensile stresses were approximately

2100 psi. The bending stresses were too large: if prestressed to resist the tensile stress provided, then the compressive stress allowances would be exceeded. Increasing the CWP wall thickness was forsaken because of the higher axial tensile stresses that would result. For this reason, this design configuration was not considered in greater detail for cold water pipe concept evaluation.

b. Articulated Wall Joints.

The articulated CWP consists of many short segmented pipes connected at joints. The segmental attachment joints can have axial, lateral, and rotational stiffnesses (zero rotational stiffness would correspond to the universal joint). In this case, the short segments of pipe respond essentially as rigid bodies, with their bending stiffness contributing little to the overall deformation of the pipe. The pipe dynamic deformation is dominated by the instantaneous axial tensions and the rotational stiffness of each joint.

Rotational-stiffened and center-supported universal articulated wall joint CWP configurations were investigated and costed. Both lightweight and mediumweight concrete CWP's were investigated for the rotationally stiffened CWP configurations. Lightweight concrete was investigated for the center-supported configuration because the structural strength requirements of the walls are not as great as those required for the rotational stiffened, wall-supported configurations.

A perspective view of rotational stiffened articulated joint is shown in Figure 3.3-11. This type of joint has been proposed by APL/JHU for use for the OTEC pilot plantship. The joint is an interlocking (thus

Figure 3.3-10. Envelope of Maximum and Minimum Bending Stresses, Brazil

simplifying deployment procedures) ring-stiffened type. The upper section rotates approximately 15 deg to lock within the lower section. The rotational stiffened joints are required at 50-ft centers.

The rotational stiffness is dominated by the geometry of the joint and the elastomer bearing pad (elastomer sheets bonded on metal shims) shear stiffness. This arrangement has been used to simulate a flexible connection for nozzle seals for missile fueling and for the connection between ocean risers and

the stationary blowout preventer valve stack anchor on the ocean floor; the blowout preventer has a 10-deg rotation capability. The seals and bearing pads will require extensive testing and development to determine the elastomer and shim thicknesses for this application.

The rotational stiffened joint is extremely rotationally limited; pad shear deformation of approximately 7 in. results for one-deg rotation, corresponding to a shear strain of 175 percent on the APL/JHU pad. This is within the capability of the pad, but exceeds the allowables recommended by bearing pad suppliers for long-term use.

The exact rotational stiffness of this type of joint was not determined for the conceptual evaluation of this CWP configuration. The analysis was based on bounding the CWP stressed by inputting upper and lower values for

Figure 3.3-11. Rotational Stiffened Articulated Joints

joint stiffness. The upper value of 3.0×10^9 ft-lb/radians was obtained from APL/JHU. The lower values of 3.1×10^7 ft-lb/radians was calculated on the assumption that the joint center of rotation remained fixed and that all pads deformed the same amount.

The envelop of maximum and minimum bending moments for the wave spectra and currents at Puerto Rico (the spar platform) and for Brazil (the barge platform) are presented in Figures 3.3-12 and 3.3-13, respectively. Results for operational and hurricane seas are shown. Similar outputs were obtained for the mediumweight concrete CWP configurations. The higher joint stiffnesses resulted in the largest design moments.

Figure 3.3-12. Envelope of Maximum and Minimum Bending Moments Jointed Lightweight Concrete CWP, Puerto Rico Wave Spectra and Currents

MAXIMUM AXIAL LOAD AT
PLATFORM/CWP ATTACHMENT
POINT = 8,510,000 LB

Figure 3.3-13. Envelope of Maximum and Minimum Bending Moments Jointed Lightweight Concrete CWP, Brazil Wave Spectra and Currents

The design moments and axial tensions for the CWP configurations are presented in Table 3.3-12 for the lightweight concrete and in Table 3.3-13 for the mediumweight concrete. The CWP walls were proportioned for the higher joint stiffness value. The nominal wall thicknesses varied from 6 to 10 inches. The large predicted joint rotations for the extreme conditions will require additional analysis.

A detailed stress analysis is required of the articulated joints' details. The analysis for effects of the eccentricity of the loads (i.e., spreading loads) at the joints was approximated during this phase of the study. The resultant section thicknesses determined were greater than those determined by APL/JHU. The increased section thickness was used as the bases for costing of the CWP configurations. The total volume of concrete at the joints represents approximately 45 percent of the entire CWP.

Table 3.3-12. Lightweight Concrete CWP Design Summary

- ARTICULATED WALL JOINTS – $K_{\theta} = 3.0 \times 10^9$ FT-LB/RADIAN

PIPE SECTIONS	BRAZIL (BARGE)		PUERTO RICO (SPAR)	
	OPERATIONAL	EXTREME	OPERATIONAL	EXTREME
UPPER				
BENDING MOMENT (FT-KIPS)	4,885	46,095	10,090	56,415
AXIAL TENSION (KIPS)	3,910	7,704	4,130	4,450
JOINT ROTATION (DEG)	0.1	0.9	0.2	1.1
LOWER				
BENDING MOMENT (FT-KIPS)	4,850	37,740	7,220	68,250
AXIAL TENSION (KIPS)	585	1,280	680	695
JOINT ROTATION (DEG)	0.1	0.7	0.1	1.3

Table 3.3-13. Medium Weight Concrete CWP Design Summary: Extreme

- ARTICULATED WALL JOINTS – $K_{\theta} = 3.0 \times 10^9$ FT-LB/RADIAN

PIPE SECTION	BRAZIL (BARGE)	PUERTO RICO (SPAR)
UPPER		
BENDING MOMENT (FT-KIPS)	38,335	40,725
AXIAL TENSION (KIPS)	10,120	8,671
JOINT ROTATION (DEG)	0.7	0.8
LOWER		
BENDING MOMENT (FT-KIPS)	45,870	57,585
AXIAL TENSION (KIPS)	1,695	1,265
JOINT ROTATION (DEG)	0.9	1.1

The center-supported, articulated CWP configuration consists of 100-ft pipe sections connected at the universal joints. The concrete sections are supported (e.g., Figure 3.3-14) by an eight-member truss structure (spiders). The truss structure was sized to provide the anticipated ring stiffness and transfer of loads from the CWP walls to the steel center post. The sections are under their own weight between joints, with all tensile load resisted by the center post. The CWP wall sections are stiffened at the support spiders to resist the kick loads resulting from changing the direction of the loads. The CWP internal diameter was increased to 31 ft to improve the flow losses resulting from the support spiders.

Figure 3.3-14. Rod-End Bearing Universal Joint

Four types of center-supported universal joint support details were investigated: rod-end bearings, ball and socket, flexures, and chain linkage. Sea water corrosion protection for the universal joints was not investigated in great detail during this phase of the study. Some means of encompassing the joint within a corrosion protected environment (e.g., coating with neoprene) could be provided. The cost of corrosion protection for the center post and support spider was calculated similar to that of the steel CWP's.

Each joint was analyzed for the intended loads. The maximum predicted rotation of the joints was 3.6 degrees at Puerto Rico and 2.4 degrees at Brazil. All joints have a capability of 5-degrees rotation.

The rod-end bearings were used as the baseline configuration for costing of the center supported CWP concept. A layout of the rod-end bearing (trade name Monoballs or Dyflon bearings) universal joint is shown in Figure 3.3-14. The Monoball is an off-the-shelf piece of hardware. Because the bearings are spherical, joint rotation is not limited to two directions. The supplier presents convincing design data that tends to verify acceptable bearing stresses, pressure-velocity factors, wear rates, and fatigue life that make the Monoball attractive for the CWP applications. Monoballs with a 5-million pound load capacity were quoted at a cost of approximately \$6,000 each. The quoted cost was increased by 50 percent to include provisions for corrosion protection.

The monoballs require no lubrication of the sliding surfaces; they have a plastic alloy liner (a proprietary material known as Dyflon) to improve sliding capacity. The material is placed in a recessed area to improve its service life characteristics.

The remaining center-supported universal joint details investigated are shown in Figures 3.3-15 through 3.3-17. If a concrete, center-supported CWP is chosen for preliminary design, these configurations will be studied in greater detail during the component design tradeoff studies.

Figure 3.3-15. Ball and Socket Universal Joint

FLEXURE DETAIL

Figure 3.3-16. Flexured Universal Joint

Figure 3.3-17. Chain Linkage Universal Joint

3.3.4.4 Complaint Design

a. Cable Reinforced Elastomer. The cable reinforced elastomer pipe concept was originally proposed by Goodyear Aerospace Corporation. The proposed concept is shown in Figure 3.3-18. The selected elastomer was neoprene; the wall is reinforced with high-strength, closely spaced wire ropes embedded in the neoprene. This type of wall construction has shown good resistance to the degradation of sea water immersion (see subsection 3.1.3.5) and is similar in construction to standard conveyor belting. Pipe walls would be constructed of 6-foot wide belts, longitudinally spliced by vulcanizing at pre-formed, scarfed joints. The splices would be made at the pipe fabrication contractor's facilities. The pipe section would be constructed in 50-foot lengths ready for shipment and assembly at the designated shore site.

Figure 3.3-18. Cable Reinforced Elastomer Concept

Compliant pipe cross sections, when subjected to the lateral hydrodynamic pressures imposed on the CWP, will experience ovaling and radial contraction deformations. The pipe walls were circumferentially stiffened with closely spaced external structural rings to minimize these deformations. Large deformations of the CWP wall sections could result in unstable internal flows. Ring sizes were determined from a strength analysis (i.e., lateral pressure buckling) requirement. Ring attachment would be with through-bolts through the belting material backed up by neoprene-coated beveled plates on the inside of the pipe (see Figure 3.3-19).

As presently configured, the stiffener rings and backing plates would be fabricated from structural steel sections and spaced at 6-foot centers. The ring sizes are standard tubes 7.25-in. outside diameter at the top of the CWP, and vary in cross section intervals along the length of the pipe to 5.25 in. outside diameter at the bottom.

If this concept is selected by NOAA/DOE for preliminary design, alternative ring materials which are not subject to sea water corrosion (e.g., FRP) will be investigated. Incorporation of transverse cord fabric plies within the basic wall material matrix to increase the pipe hoop tension and shear stiffness will also be investigated as alternatives to closely-spaced rings during preliminary design.

Analysis of the pipe cross-sectional deformation due to lateral pressures indicated that the maximum area reduction was less than 0.75 percent. This could be further minimized by closer ring and cable spacing and larger ballast weights; however, conceptual analysis indicated that the predicted cross-sectional changes would not result in unsteady internal flow (Venturi tube effects) or in excessive frictional losses. Initial conclusions were that a diameter change of at least 10 percent would be required to develop significant unsteady internal flows. For the preliminary design, internal and external flow dynamics would be considered in greater detail.

The global structural analysis (both static and dynamic) of the cable reinforced elastomer pipes were performed using the TRW CWP computer model for the plant operational environmental conditions (sea state 5 plus currents). The cross sections were proportioned (i.e., selection of cable diameter and spacing) and ballast weights sized such that the CWP walls would remain under tensile loads at all times. This design requirement would ensure that the pipe cross section would remain stable (not buckle) and could continuously supply cold sea water during plant operations. Frequency curves from the environmental package indicate that 10-ft significant wave heights (sea state 5) would be exceeded less than two percent of the time. The amount of pipe wall buckling is closely associated with its predicted fatigue life. A safety factor of 2.0 was used for comparison with the ultimate cable breaking strength.

A statical analysis for operational currents only indicated the CWP required a gimbal platform hull attachment. The analysis indicated large bending stresses near the upper sections of the CWP with a fixed platform attachment. The predicted compressive bending stresses were greater than the tensile stresses (indicating buckling of the pipe section) for an

economic combination of cable area and ballast weight. It was more cost effective to add a gimbal attachment at the hull.

A series of computer runs were made for various combinations of cable size and spacings and ballast weights. Analysis of the original pipe wall section proposed by Goodyear, 0.1875-inch diameter cables at 3-inch centers embedded in 1-inch thick neoprene walls, proved unsuccessful when subjected to the previously stated design requirements. The pipe wall configurations determined from the analysis consist of 0.5-inch diameter cables at 3-inch centers embedded in 1-inch thick neoprene. The complete CWP is presented in Figure 3.3-19. The ballast weight was 1.75 million pounds. The envelope of maximum and minimum stresses for this pipe configuration is presented for Brazil and Puerto Rico in Figures 3.3-20 and 3.3-21, respectively. The tensile loads are greater than the predicted bending stresses for all positions along the pipe.

Figure 3.3-19. Cable Reinforced Elastomer, Structural Details

MAXIMUM AXIAL LOAD AT
PLATFORM/CWP ATTACHMENT
POINT = 2,520,000 LB

Figure 3.3-20. Envelope of Maximum and Minimum Stress Cable Reinforced Elastomer, Brazil Operational Wave Spectrum and Currents

MAXIMUM AXIAL LOAD AT
PLATFORM/CWP ATTACHMENT
POINT = 2,6000,000 LB

Figure 3.3-21. Envelope of Maximum and Minimum Stress Cable Reinforced Elastomer, Puerto Rico Operational Wave Spectrum and Currents

Investigation of the CWP behavior for the extreme sea states was not attempted during this phase of the study. This analysis, which would include an investigation of the nonlinear behavior of the CWP cross-section after compressive stresses in the cables were predicted, should be investigated during the preliminary design of this concept. A prediction of excessive amounts of compression would lead to a general instability of the pipe cross section (i.e., a crimping failure).

The amount of cross-sectional crimping is closely associated with the fatigue life analysis for this configuration. Fatigue failures of wire in wire ropes result from the propagation of small cracks when the cables are repeatedly bent. This normally occurs when the ropes operate over comparatively small sheaves or drums. The repeated bending and straightening of individual wires as the ropes move can cause fatigue, and eventually failure. No fatigue data exists for CWP type loads. Fatigue life predictions will have to be investigated in greater detail during preliminary design.

The steel ring-stiffened CWP was used as the baseline configuration for costing, risk evaluation, and ranking. The results of the conceptual evaluation indicate that this configuration is feasible for OTEC applications.

b. Bell Design. The proposed Bell design for the CWP is a unique design that has some distinct advantages, but also some disadvantages and some unresolved problems.

The design consists of a double rolled fabric cylinder with the two walls connected by a series of radial* webs to form longitudinal compartments extending the length of the pipe. These compartments are then pressurized with water to place the fabric in tension, which maintains the pipe's shape. As a result of the pressurization, the inner and outer walls are pushed out to form a series of circular lobes between the webs.

*To increase the lateral pressure buckling capability of the pipe, these webs are being redesigned to form angular compartments. The trusslike configuration should increase the shear stiffness and through the buckling capability.

The fabric consists of neoprene rubber reinforced with nylon cloth stretched to about 7 percent strain by the internal pressure. The ultimate strain capability of the fabric is 40 percent.

Individual sections of the pipe are joined by metal diaphragms that extend between the inner and outer walls of the pipe. These diaphragms are perforated, and the perforations are sized to control the dynamic response of the columns of water in response to the heave motion. In response to heave motion, it is expected that more of the 200 million lbs of water filling the pipe will remain fixed and the differential motion between the fixed water and the barge will be taken up by stretching at the top portion of the pipe.

The principal advantage of the design is that it can be depressurized for weather conditions that exceed operational limits. When unpressurized it becomes very flexible and can easily accommodate to the wave and current motion. However, its behavior under the associated heave motion is unclear.

The cylinder is relatively easy to transport and deploy in the unpressurized condition. It is also possible to retrieve the pipe for repair by disconnecting the ballast. In common with all elastomer-based designs with large strain capability, this configuration can sustain momentary local buckling in the operational condition without a destructive failure.

Calculations were run using the TRW Cold Water Pipe computer model for both the Brazil and the Puerto Rico operational conditions. The results showed a maximum bending strain of 0.3 percent which is well within the capability of the wall material. The results also showed a very rapid buildup of bending stress at the ends of the pipe where it joined the ship and the ballast, however, and this rapid buildup indicates large shear forces at these locations, as expected. The capabilities of the pipe to sustain these large shear forces is not clear; the TRW computer model does not include shear deformations so that the behavior at these critical locations is not accurately modelled.

The principal disadvantage of the design is that it is an active system requiring continued pumping to make up the inevitable leakage. Successful operation also depends on maintaining water, tightness of literally miles of seams that are undergoing constant deformation.

Additional problems are the behavior under heave excitation and at the connections with the barge and ballast, and the stress systems behavior of the nylon/rubber fabric subjected to large strains for a period of 30 years.

Finally, the effect of the external scallop-shape on vortex shedding and the response of the flexible cylinders to its excitation is not known.

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3.4 CWP/PLATFORM TRANSITION ANALYSIS

The CWP/Platform interface poses two problems: the first is to provide a seal for the cold water passing between the CWP and the platform. The seal has the same performance requirements as the CWP: pressure/flow losses, thermal insulation, maintainability, and life cycle costs.

The second problem is to transmit loads across the interface and to withstand the relative motions between the platform and the CWP. The loads will vary from 100 percent of the weight of the CWP to 0 percent. The 0 percent weight applies when the CWP is self-supported or bottom-mounted. The relative motions will vary from zero for a cantilever or rigid connection, to large angles where the connection is made by a compliant section or other type of transition that allows relative motions.

The transition can usually be divided into two types: those that support the weight of the CWP and those that do not. The transitions that support the CWP weight are fixed to the platform in some, if not all, of the degrees of freedom of motion. The CWP's with upper ends terminating in these connections are not restrained at any other point along their length: they hang down and move freely below the platform. The transitions that do not support the CWP weight are those that are fixed to the bottom, or that have self-supporting flotation collars. Where the CWP is attached to the sea bottom, the applicability is only to the stationary OTEC platform.

Where the platform is in the grazing mode, the transition will transmit the weight of the CWP or withstand the relative motion between the platform and CWP. If the CWP is supported by the platform, loads up to the dynamic reaction seen at the top of the CWP will be transmitted through the transition. The transition and seal may also be required to withstand some amount of relative motion between the platform and the CWP if the structural and dynamic analysis require this capability to reduce the stresses on the CWP. The self-supporting CWP transition is discussed below in the decoupled case.

The stationary mode platform transition will solve the same problem as the grazing plant if the CWP is supported from the platform. A different set of problems must be solved if the CWP is attached to the bottom; the bottom-mounted CWP will either decouple the loads and motions through the interface or transmit all or part of the stationary plants mooring system loads if a hard connection is used. Making the bottom-mounted CWP the mooring system for the platform requires utilizing tension-leg or taut line mooring techniques and analysis. The mooring loads of keeping the platform stationary will then be added to the other loads.

In the decoupled case, the CWP is self-supported by buoyancy, but the transition must still provide the seal between the CWP and the platform. This transition must also withstand the relative motions between the platform and the top of the CWP.

3.4.1 Approach

Generic types of joints were first investigated and listed, then discussed and screened by the review board with only very brief conceptual work accomplished. The board determined joint applicability and the configurations to pursue during the concept evaluation phase of the work.

Conceptual designs of the selected generic types of the load-supporting transitions were developed based on the loads and relative motions to be expected from the hydrodynamic and structural analysis performed on the CWP candidates. Preliminary structural analysis was performed for each generic type of transition. Three loads were used to represent the range of loadings and motions expected from the CWP's under investigation. Preliminary drawings were prepared of the conceptual transition designs; these were priced to provide cost curves for each over the range of expected loads.

Assessment of the advantages and disadvantages of these designs were then performed in terms of the ROM costs; ease of deployment, maintenance, fabrication, repairs and detachment; and the applicability to the two platforms configurations.

The advantages-disadvantages assessment lead to the preliminary determination of suitability of specific transition designs to CWP candidates, as well as the decision on which transition best suited each platform design.

3.4.2 Results

3.4.2.1 Generic Types

The description of generic types were developed for the screening board review as follows:

- Hinged at the platform and supporting the CWP. This type transition decouples the platform from the CWP in two degrees of freedom, pitch and roll, while coupling remains in heave, surge, sway, and yaw, and supports the weight of the CWP.
- Hinged with heave compensation. The same as hinged, above, except it also decouples the motion of heave.
- Cantilever support of the CWP. This transition coupled all motions and is a rigid connection between the CWP and platform.
- Self-supported CWP. The CWP would either be bottom-mounted or have provisions for self-supporting flotation collars. This generic type also includes those CWP's that serve as part or all of the platform mooring system.

3.4.2.2 Preliminary Screening Results

The initial analysis of the CWP's hydrodynamic motions and structural dynamic loads showed that with a CWP rigidly attached to a platform with a cantilever transition, the stresses developed in the pipe were extremely high. This would require the CWP to be a massive structure to withstand such loads. The analysis also showed that the additional dynamic axial loadings due to heave were small in comparison to the bending stresses.

Such preliminary results were sufficiently conclusive to eliminate further investigation on hinged joints with heave compensation as not being required. Further, cantilever transitions were eliminated because of the high stresses developed in the CWP.

The screening board members eliminated self-supporting CWP's, except for the bottom-mounted type, because of the problem of relative motions between two floating bodies at sea, and the difficulties associated with maintaining the seal under these conditions. Mild sea conditions make decoupled transitions very attractive and probably inexpensive, but downtime caused by relative motion between the CWP and the platform would probably occur long before the platform equipment shutdown point was reached. If this type of transition was designed to operate through plant equipment shutdown sea states, much of the attractiveness of simplicity and low cost would disappear.

At the end of the preliminary screening process two generic transition types were left for the concept selection phase analysis: hinged transitions for both grazing and stationary platforms, and transitions associated with the stationary platform and bottom mounted CWP's.

3.4.2.3 Loadings and Angular Motions of the Supported CWP's

Hydrodynamic and structural analysis on the various CWP configurations provided the range of loads and angular deflections the hinged transition would be required to transmit to the platform. These loads varied from maximum dynamic axial load of 20.5×10^6 to 2.25×10^6 lb. The maximum deflection varied from 16 to 25 deg, depending on the submerged weight of the CWP.

The monolithic steel CWP provided the maximum loadings of 20-1/2 million lb. The multi-jointed steel CWP conceptually placed a lower range of steel pipe weights at a static wet weight of about 6-1/2 million lb.

Similarly, conceptual designs of medium-weight concrete placed the upper limit of this material concept at 15×10^6 lb maximum dynamic axial load; a lower limit of about 3-1/2 million lb static weight was represented by light-weight concrete.

The FRP cold water pipe loadings varied from a maximum of 6.5 million lb axial dynamic load for solid pipes downward. The lower limit was not established by the weight of such CWP's as sandwich FRP and polyethylene,

but by the amount of stabilizing weight required. The stabilizing weight is needed to maintain the CWP at a small angle to the vertical direction during design current conditions. This weight has been calculated to be from 1 to 2 million lb. The lower limit on all hinged joint loads was taken therefore at 2 million lb.

Conceptual designs and analysis were performed for various generic types with the above loads on the hinged CWP/platform transitions. Twenty and one-half million lb was the upper limit load and corresponded to the heavy steel CWP; 7.6 million was used for the medium-load corresponding to concrete, with 2.25 million for the lower limit FRP. All hinged transitions were capable of rotating freely through 20 deg from the vertical. Once analysis began, it was evident that differences of a few degrees of rotation would depend more on the supporting structures than on the transition design.

3.4.2.4 Conceptual Designs of Platform-Supported Hinged CWP Transitions

Five types of conceptual transition designs were developed for supporting the CWP. For each of these types, preliminary structural analysis was performed to size the structure necessary to transmit the load expected from each of the CWP materials mentioned in subsection 3.4.2.3.

The attachment point of the CWP to the platform was assumed to be at the same location as shown in the preliminary designs of the barge and spar. The location of this point should be investigated further since it not only affects the hull in providing clearance for the CWP movement, but also impacts the seal and transition design. The location used places the majority of the transition sections for the barge and all of the transition sections for the spar below the sea's surface.

Of the conceptual types as configured, all but the last are applicable to the barge only. Additional connecting or transition pieces not designed at this time would be required on these designs to adapt them to the spar OTEC platform. The last design could be utilized equally well on either platform. The additional transition requirement is due to

the central column size in the spar platform. It would be necessary to launch the spar with the transition in place; these CWP's would have to be deployed, then attached to the spar. The central column has insufficient diameter to deploy the pipe from the platform through the central column.

All the analyses and designs were completed to the same degree at this phase in the design process which allowed comparison of the ROM costs on all the conceptual types.

The CWP supporting transition conceptual types investigated were:

- Gimbal joint with compliant seal and with a ball seal
- Biplanar flex joint
- Socket joint
- U-joint
- Flotation ball and socket.

a. Gimbal. The gimbal joint, Figure 3.4-1, was chosen as one of more straightforward methods of decoupling pitch and roll. This type joint is used extensively in the marine industry where an object is required to be decoupled from these ships motions in the plane of the object. Gimbals are used in sizes from small objects, such as a compass, to the largest known, the Glomar Explorer gimballed heavy lift system and derrick. The Glomar Explorer heavy lift system is capable of decoupling weights up to 20 million lb.

The sealing function of the transition was treated in two ways: by utilizing a flexible compliant member between the gimbal and the hull, and by placing a ball surface inside the gimbal to provide the required seal. Both of these seals would be designed to be serviceable or replaced during the life of the OTEC 10/40 barge should it be necessary.

The gimballed transition joint provides an additional advantage in that it could be used as the deployment base on the barge. By using the gimbal as a support for the assembly structure, some of the problems of connecting large, heavy pieces of the CWP at sea are reduced (discussed in more detail in subsection 4.2).

Figure 3.4-1. OTEC 10/40MW' CWP/Hull Transition Gimbal

Placing the joint below the waterline introduces the problem of submerged bearings and their lubrication, maintenance, or replacement during the life of the program. The use of bearing material that requires no lubrication (uses either seawater as a lubricant or a pressurized lubricating oil system) will have to be investigated. The type of bearings or bearing material most suitable for each load is also an area of uncertainty. Additionally, as mentioned above, the use of this type of transition for the spar platform requires an additional connection between the transition and the CWP once the CWP is in place below the spar.

b. Biplanar Flex Joint. The biplanar flex joint, Figure 3.4-2, is a modification of the gimbal joint, and is also an extrapolation of the same joint currently used for the lower universal on the biomass kelp farm. The biomass structure supports a 2-ft diameter, 1500-ft long. This design has proven an acceptable, workable solution in the 2-ft diameter pipe.

Some advantages over the gimbal are offered by this design: because of the proximity of the load transmitting bearings and the support design, the bending moments through the pins were reduced allowing smaller bearing pins to be used for equivalent CWP weights.

The disadvantage of the gimbal and the submerged bearing is also true for this design. Also, the seal of the biplanar joint is not as easily serviced and is more susceptible to damage because it is housed within the joint. Finally, this design cannot be used as the supporting structure for deployment of the heavy CWP's from the barge platform.

For all these reasons, we feel that this design does not offer sufficient advantages to make it more viable than the gimbal transition. ROM cost differences discussed later in this section also contributed to the final decision.

GENERAL NOTES:

1. FRP = FIBER-REINFORCED PLASTIC
2. PINS TO HAVE YIELD STRENGTH OF 60 KSI - ALL OTHER STEEL TO HAVE YIELD STRENGTH OF 45 KSI.
3. THIS DRAWING PORTRAYS CONCEPTUAL DESIGN ONLY. DO NOT FABRICATE OR MANUFACTURE TO THIS DRAWING.

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OTEC 10/40 MW-CWP/HULL TRANSITION
BIPLANE FLEX JOINT

Rev	By	Date	Scale	Tolerance Unless Otherwise Noted Fraction: 1/16" Angular: 1/2°
DWG	C. MOSKOWSKI	2-17-79	NONE	
ENGR	C. MOSKOWSKI	2-22-79		
CHKD	C. MOSKOWSKI	2-27-79		
APP	C. MOSKOWSKI	3-17-79		
DES	C. MOSKOWSKI	3-17-79		
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1/16" = 0.0625" 1/2° = 0.873°

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Figure 3.4-2. OTEC 10/40 MW CWP/Hull Transition Biplaner Flex Joint

c. Socket Joint. The socket joint, Figure 3.4-3, has been proposed in several forms for possible use in supporting the CWP of the various OTEC projects. The conceptual design shown utilizes a bearing surface that also acts as the seal.

Since this design for the barge is anticipated to be in a well or moon pool open to the main deck, it offers the easiest maintenance procedures of any of the designs considered. A heavy lift A-frame would be used to lift the whole CWP and attached portion of the transition clear of the water for maintenance/replacement of the elastomeric bearing pads. As with the gimbal, this type of transition also provides a base for the deployment structure necessary for heavy CWP.

One of the problems associated with the socket joint is the material life of the bearing surfaces under these conditions. How often these pads need replacement will have to be investigated. The bearing pads are held against the shell by gravity and the differential pressure between the cold water sump and the sea. No positive means keeps the joint from unloading in the vertical upward direction. This may, under some environmental conditions not investigated to date, cause impact loads that could produce failures of the structure and sealing surface; the design of this transition would have to ensure against this failure.

d. U-Joint. The U-joint, Figure 3.4-4, provides only one large bearing to be maintained which for application to the barge platform could have the bearing above the waterline to eliminate the problem of the submerged bearings associated with the gimbal and biplaner flex joints. As with the previous three joint designs, however, this design offers no better solution for applicability to the spar platform.

The major problem associated with this design is the seal. Having to place the seal many feet away from the U-joint and the imposed 20 deg of rotational freedom makes this seal most difficult. Initially envisioned as a compliant member, the seal would have to accommodate about 15 or more feet of movement, depending on its exact location. This is probably not feasible.

Figure 3.4-3. OTEC 10/40 MW-CWP/Hull Transition Socket Joint

Figure 3.4-4. OTEC 10/40 MW CWP/Hull Transition: U-Joint

An alternate type of seal that may be possible but not shown is the sliding rigid seal formed by a portion of a sphere. This type seal would be a massive structure and undoubtedly drive the cost of the U-joint transition very high. It may also provide many fabrication and deployment problems not associated with other designs.

e. Flotation Ball and Socket Joint. The flotation ball and socket joint, Figure 3.4-5, was proposed as a possible solution to some of the problems associated with the preceding joint designs. This design could be used as easily for the spar as for the platform and provides perhaps the most simple attachment procedures. The maintenance or replacement of the seal bearing pads would also be relatively easy. The loadings developed may prove to be smaller because the force would be upward and need only provide the sealing force necessary. Also, by providing means of changing the buoyancy through flooding and deballasting a portion of the flotation ball, this joint could easily be decoupled from the platform should the need arise.

This transition is a combination or cross between the supporting CWP transitions described above and the bottom-mounted CWP discussed in subsection 3.4.2.5. Generically, it would be classed as a self-supporting type of transition, but it does not have the problem with this class of relative motions between decoupled floating bodies.

The problems associated with this transition are similar to those associated with the socket joint. The life of the seal/bearings in seawater must be addressed. The design must also ensure that no environmental conditions could cause complete unloading of the connection that would result in joint-destroying impact loads. One additional disadvantage to this type joint results from the large diameter ball required for heavy CWP and the fabrication costs associated with the larger machined surfaces. The ROM costs developed indicate that this transition will be the most costly of the five presented above.

Figure 3.4-5. OTEC 10/40 MW CWP/Hull Transition Flotation Ball and Socket

3.4.2.5 Conceptual Design Bottom Mounted

For the stationary spar platform, most of the conceptual designs for the supported CWP presented above have the drawback of complex deployment and attachment problems. By using a bottom-mounted CWP, some of these problems may be reduced or otherwise compensated for. The generic types of bottom-mounted CWP are:

- Top moored with buoyant collar
- Bottom connected serving as tension-leg mooring for the platform
- Riser type
- Bottom moored with buoyant collar.

The top moored type would be similar to the supported CWP except the support would be supplied by buoyant sections or a buoyancy collar. Mooring lines from the top of the CWP would be used to hold the pipe in the desired location and a transition piece would be used to connect to the platform.

This type was not selected for conceptual design because:

- There is no distinct advantage over the supported transitions of 3.4.2.4.

- The mooring system on top of the CWP duplicates the mooring system necessary for the platform.

- The relative motion of the platform and the CWP with a 4000-ft water depth catenary mooring system may be large.

The CWP that is bottom-connected and serves as the mooring system for the platform is the form of a tension-leg mooring being investigated not only by Gibbs and Cox, but by the offshore oil industry as well. Investigation of this type of CWP depends strongly on the platform design.

Tension-leg mooring systems are designed to hold the platform at a given position. The CWP, which would also serve the function of the mooring system, would have to be designed to withstand these other forces. It is expected that the mooring forces will be at least as large as current investigations show for the CWP alone, if not a good deal larger. This

would, by itself, place an unfair cost penalty on this CWP concept. Further, the design of a tension-leg mooring system should parallel the platform design to tradeoff design parameters between the two systems. For these reasons, no further design work was performed on this concept.

The riser type CWP has been proposed and offers some advantages in that it is currently used for deep water drilling by the offshore oil industry, which places it in the realm of current technology for the sizes utilized by drilling companies. The 30-ft diameter CWP for the OTEC 10/40 platform is larger than can be deployed from the spar OTEC platform, unlike current offshore procedures, and is much larger than currently used. Complex double swivels and slip joints would be needed for this concept to work. These slip joints and swivels would require extensive design work equivalent to the work necessary for the supporting joints discussed in subsection 3.4.2.4. The problem of a deployment vessel capable of handling these sections will be as great as that expected from the supported CWP transitions. For these reasons, the riser type was not investigated further.

The bottom moored type with buoyant collar was investigated for comparison with the supported CWP. This concept offered the greatest advantages of all the bottom mounted concepts, and the closest comparison to the supported concepts. A compliant section would be used between the CWP and the hull. Relative motion between the CWP and the platform must be small for the compliant transition to be feasible. This requires the spar platform to maintain relative position above the CWP upper end. This type of bottom-mounted CWP is shown generically in Figure 3.4-6.

This transition offers many advantages when the relative separation of the CWP and the platform is kept small. The need for hinged connections is eliminated and the angular motion is absorbed by the compliant section. The tension required to keep the CWP vertical in this concept is no greater than the load associated with the supported CWP's. The compliant section can be manufactured in a manner similar to the total CWP proposed by Goodyear.

Figure 3.4-6. Bottom-Mounted CWP

The bottom connecting link can be off-the-shelf materials such as chain, cables, or pipe. The clump or bottom anchor can be dead weight or an anchored structure with pilings. The anchoring structure decision would ultimately depend on which concept proved most advantageous in terms of cost and installation procedures. This concept requires a clump weight similar to the stabilizing weight required for some of the CWP concepts.

3.4.3 Conclusions

The CWP/platform transition design ultimately chosen will be greatly affected by the material and the design of the CWP itself. The hinged transition will be much lighter for CWP's that have less submerged weight than those of dense heavy material. Conceptual analyses indicate that the lower limit of the load the transition will experience is given by a stabilizing weight of 2 million lb which will be necessary on neutrally buoyant CWP's. Table 3.4-1 presents a summary of the transitions considered in the conceptual design phase of this work.

The gimbal and socket transitions of the hinged joints provide the best overall solutions to the transitions that support the CWP. Also, the bottom-mounted tension CWP with buoyant collar and compliant transition offers the best solution of these generic types. The flotation ball and socket transition offers some unique solutions and some distinct advantages over the other transitions. If the cost figures generated for this concept

Table 3.4-1. CWP/Platform Design Assessment

DESIGN	MAJOR ADVANTAGE	MAJOR DISADVANTAGE	APPLICABILITY		
			CWP CONFIGURATION	BARGE	SPAR
GIMBAL	COST AND LARGE LOAD CAPABILITY	MAINTENANCE AND SUBMERGED BEARINGS	ALL, IF BEARING PROBLEM CAN BE SOLVED	YES	YES, WITH ADDITIONAL CONNECTION
BIPLANE FLEX	SMALLER BEARING/ PIMS OVER GIMBAL	SAME	GIMBAL/BETTER	GIMBAL/BETTER	GIMBAL/BETTER
SOCKET	SIMPLE AND MAINTAINABLE	LIFE OF BEARING PADS AND POSSIBLE IMPACT LOADINGS	ALL	YES	YES, WITH ADDITIONAL CONNECTION
U-JOINT	LOAD BEARING PORTION OUT OF WATER	SEAL DESIGN	LIGHT	YES	NO
FLOTATION BALL/SOCKET	DEPLOYMENT AND CAN BE UNCOUPLED	COST AND POSSIBLE IMPACT LOADS	LIGHT (HEAVY LARGE BALL)	YES	YES
BOTTOM-MOUNTED TENSION BUOYANT WITH COMPLIANT TOP	SIMPLE AND CAN BE UNCOUPLED	RELATIVE MOTIONS TAKEN BY COMPLIANT PIECE	LIGHT	NO	YES

during this phase could be reduced and the question of impact loadings answered, this transition should be considered further. Because of the uncertainty of the submerged bearings in the gimbal transition, the socket joint was used in all cost comparisons.

The ROM costs generated during this phase of work are shown in Figure 3.4-7, with backup estimates provided in Appendix G.

Figure 3.4-7. CWP/Platform Transition Cost Versus Pipe Axial Loads

3.4.4 Concerns

Several questions and concerns that have a major impact on the CWP/platform transition design remain for investigation during the next phase of work. The answer to these questions will effect the transition selection process as well as the cost figures generated during the preliminary design process. These concerns are:

- Connection or connecting piece for the CWP that are keel-hauled; how will they be connected to the transition?

- Are there any environmental conditions that would cause unloading of transitions that are not fixed to the platform, such as flotation ball joint and socket joint?

- How to maintain the separation of platform and top of the CWP for bottom-mounted CWP concepts to a value for which the compliant section will work?

- Can the costs be reduced on the flotation ball concept to make it more viable, or would the costs associated with making the other concepts disconnectable approach this type of transition?

3.5 REFERENCES

- 3-1 Gurney, T. R. and S. J. Maddox, "A Re-analysis of Fatigue Data for Welded Joints in Steel," Report E/44/72, The Welding Institute, Cambridge, England, January 1972.
- 3-2 TRW IOC 5515.2.78-537, "OTEC Cold Water Pipe, Design Concept 9 - Some Materials Engineering Considerations," J. S. Hicks to M. P. Bianchi, 16 November 1978.
- 3-3 "Rules for Building and Classing Steel Vessels," American Bureau of Shipping, 1976.
- 3-4 "Rules for the Design, Construction, and Inspection of Offshore Structures," Det Norske Veritas, 1977.
- 3-5 LaQue, F. L., *Behavior of Metals and Alloys in Seawater*, Corrosion Handbook, M. M. Uhlig, Ed., John Wiley, 1966.
- 3-6 L. L. Schrier, Ed., *Corrosion*, Newnes-Butterworths, London, 1977.
- 3-7 Gould, A. J., "Corrosion Fatigue," Institute Conference of Fatigue of Metals, Session 4, Paper 2, Institute of Mechanical Engineers and ASME, 14 September 1956.
- 3-8 TRW IOC 5515.2.78-564, "Preliminary Data on Corrosion of Steels at Depth in Seawater," M. P. Bianchi to A. B. Griffin, 29 November 1978.
- 3-9 TRW IOC 5515.2.78-446, "Neoprene Coating Summary for OTEC," M. P. Bianchi to L. A. Rosales, September 20, 1978.
- 3-10 Lennox, T. J., Jr., "Marine Electrochemical Corrosion and Control Systems," NRL Memo Report 3622, Naval Research Lab, Washington D.C., October 1977.
- 3-11 Bross, H. E. and J. A. Burgbacher, "Reinstallation of Cathodic Protection for Deep Water Structures," *Materials Performance*, 15, 14-17, 1976.
- 3-12 B. F. Brown, *Stress Corrosion Cracking Control Measures*, NBS Monograph 156, U. S. Department of Commerce, June 1977.
- 3-13 Graner, W. R. and R. J. Della Rocca, "Evaluation of U.S. Navy GRP Boats for Material Durability," 26th Ann. Tech. Conf., 1971, Rein. Plas and Comp., SPI.
- 3-14 Isham, A. B., SPI Reinforced Plastics Div., 22nd Annual Conf., Sec. 16-E (1967).
- 3-15 Owen, J. J. and R. G. Rose, *Modern Plastics*, 47 No. 11, 130 (1970).

- 3-16 Schadel, O., *Kunststoffe*, 59, No. 12, 892 (1969).
- 3-17 Pearson, L. E., and R. L. Stafford, SPI Reinforced Plastics/Composite Div., 25th Annual Conf., Sec. 4-E (1970).
- 3-18 Pearson, L. E. and A. B. Isham, *Materials Protection*, No. 9, 13 (1966).
- 3-19 Broutman, L. G. and S. Sahu, "Progressive Damage of a Glass Reinforced Plastic During Fatigue," 24th Annual Tech. Conf., 1969, Reinforced Plas. and Comp., SPI.
- 3-20 Mandell, J. F. and F. J. McGarry, "Fracture Behavior of Fiberglass Reinforced Plastics Suitable for Hull Materials," NOAA, PB-264019, MITSG 75-25, 1976.
- 3-21 Fried, N., "Degradation of Composite Materials: The Effect of Water on Glass-Reinforced Plastics," *Mechanics of Composite Materials*, Wendt, F. W., H. Liebowitz, and N. Perroue, 1970 Pengamon Press.
- 3-22 Richardson, C., et al., "Assessment of Concrete Cold Water Pipe for 40 MWe OTEC Unit," Sir Rober McAlpine and Sons, Ltd., London England, 1979.
- 3-23 "Recommendations for the Design and Construction of Concrete Sea Structures", 3rd Edition, Fédération Internationale de la Précontrainte, 1977.
- 3-24 "Guide for the Design and Construction of Fixed Offshore Concrete Structure," ACI, Committee 357, Title Number 75-72, December 1978.
- 3-25 Rivertz, Johan A., "Future of Concrete Structures in Offshore Technology," Report for Norwegian Contractors, Oslo.
- 3-26 "Prestressed Concrete Ocean Structures and Ships," Post-Tensioning Institute, September 1975.
- 3-27 Myers, John J., Carl H. Holm and R. F. McAllister, "Handbook of Ocean and Underwater Engineering," McGraw-Hill Book Company, 1969.
- 3-28 Neville, A. M., "Properties of Concrete," John Wiley and Sons, New York, 1973.
- 3-29 Sepak, E. and F. G. Rice, "Strength of Polyethylene Piping - New Insights," *Engineering Digest*, May-June, 1977.

- 3-30 Cessna, L. C., J. A. Levens, and J. B. Thomson, "Flexural Fatigue of Glass-Reinforced Thermoplastics," 24th Ann. Tech. Conf., 1969, Rein. Plas, and Comp., SPI.
- 3-31 Sauer, J. A., E. Foden, and D. R. Morrow, "Influence of Molecular Weight on Fatigue Behavior of Polyethylene and Polystyrene," *Polymer Engineering and Science*, April 1977, Vol. 17, No. 4.
- 3-32 Department of Energy, *OTEC Demonstration Plant Environmental Package*, January 1978.
- 3-33. Hove, D., W. Shih, and E. Albano, "Hydrodynamic Design Loads for the OTEC Cold Water Pipe," Science Applications, Inc. Report, SAI-79-559-LA, September 1978.
- 3-34 G. W. Jones, J. J. Cincotta, and R. W. Walker, "Aerodynamic Forces on a Stationary and Oscillating Circular Cylinder at High Reynolds Numbers," NASA TR R-300.
- 3-35 A. Roshko, "Experiments on the Flow Past a Circular Cylinder at Very High Reynolds Number," *Journal of Fluid Mechanics*, Vol. 10, 1964, pp. 345-356.
- 3-36 E. Achenbach, "Distribution of Local Pressure and Skin Friction Around a Circular Cylinder in Crossflow up to $Re\ 5 \times 10^6$," *Journal of Fluid Mechanics*, Vol. 34, Part 4, 1968, pp. 625-639.
- 3-37 V. S. Murthy, and W. C. Rose, "Form Drag, Skin Friction and Vortex Shedding Frequencies for Subsonic and Transonic Crossflows on Circular Cylinders," AIAA Paper No. 77-687, AIAA 10th Fluid and Plasmadynamics Conference, Albuquerque, New Mexico, June 27-29, 1977.
- 3-38 E. Achenbach, "Influence of Surface Roughness on the Crossflow Around a Circular Cylinder," *Journal of Fluid Mechanics*, Vol. 46, Part 2, 1971, pp. 321-335.
- 3-39 E. Szechenyi, "Supercritical Reynolds Number Simulation for Two-Dimensional Flow over Circular Cylinders," *Journal of Fluid Mechanics*, Vol. 70, 1975, pp. 529-547.
- 3-40 O. Giiven, V. C. Patel, and C. Farell, "A Model for High Reynolds Number Flow Past Rough-Walled Circular Cylinders", *ASME Journal of Fluids Engineering*, Vol. 99, Series I, No. 3, September 1977, pp. 486-494.
- 3-41 Griffin, O. M., "Flow Near Self-Excited and Forced Vibrating Circular Cylinders," *Journal of Engineering for Industry*, ASME, May 1972, pp. 539-547.

- 3-42 Iwan, W. D., and R. D. Blevins, 1974, "A Model for the Vortex Induced Oscillation of Structures," *J. Appl. Mech.*, p. 581.
- 3-43 Griffin, O. M., 1961, "A Universal Strouhal Number for the 'Locking-on' of Vortex Shedding to the Vibrations of Bluff Bodies," *J. Fl. Mech.*, 85, 3, pp. 591-606, 1978.
- 3-44 Sarpkaya, T., "Vortex Shedding and Resistance in Harmonic Flow about Smooth and Rough Circular Cylinders at High Reynolds Numbers," Naval Postgraduate School Report NPS-59SL76021, February 1976.
- 3-45 Stansby, P. K., "The Locking-On of Vortex Shedding due to the Cross-Stream Vibration of Circular Cylinders in Uniform and Shear Flow," *J. Fluid Mech.* (1976), Vol. 74, Part 4, pp. 641-665.
- 3-46 Mair, W. A., and P. K. Stansby, "Vortex Wakes of Bluff Cylinders in Shear Flow," *SIAM J. Appl. Math.*, Vol. 28, No. 2, March 1975, pp. 519-540.
- 3-47 Maull, D. J. and Young, "Vortex Shedding from Bluff Bodies in a Shear Flow," *J. Fluid Mech.*, Vol. 60, p. 401.
- 3-48 Korotkin, A. I., "The Three Dimensionality of the Flow Transverse to a Circular Cylinder," *Fluid Mechanics-Soviet Research*, Vol. 5, No. 2, March-April 1976.
- 3-49 Osbourne, A. R., T. L. Burch, and R. I. Scarlett, "The Influence of Internal Waves on Deep-Water Drilling", *Journal of Petroleum Technology*, October 1978, pp. 1497-1504.
- 3-50 Eckart, Carl "Internal Waves in the Ocean", *Physics of Fluids*, Vol. 4, No. 7, July 1961, pp. 791-799.
- 3-51 Little, T. E., J. D. Marks, and K. H. Wellman, "Deep Water Pipe, Pump and Mooring Study: OTEC Final Report," Westinghouse Electric Corp., C00-2642-3, June 1976.
- 3-52 Rosenhead, L., ed., *Laminar Boundary Layers*, Oxford University Press, 1963, p. 390.
- 3-53 Wiegel, R. L., *Oceanographical Engineering*, Prentice-Hall, 1964, p. 17.
- 3-54 Lin, T. Y., *Design of Prestressed Concrete Structures*, Second Edition, John Wiley and Sons, New York, 1967.
- 3-55 Gerwic, Ben C., Jr., "Prestressed Concrete Ocean Structures and Ships," Prestressed Concrete Institute, September, 1975.

4. FABRICATION/CONSTRUCTION/INSTALLATION

Selection of fabrication/construction/installation techniques for the CWP was an interrelated process that strongly depended on both the pipe materials and the configuration, including size, weight, segment joint design, and hull attachment method. The basic pipe design depends, in turn, on production and installation methods. A major alternative was whether to construct on-site or onshore, and included such options as the transport and installation of subassemblies. Cost and practicability of the technique (e.g., available facilities and resources) determined the final tradeoff. In many cases, transportation and installation in the ocean environment dictated the CWP design selection. In all cases, standard techniques and available facilities were utilized to the maximum extent possible.

4.1 FABRICATION/CONSTRUCTION

The basic fabrication and construction techniques discussed in the following sections were dictated to a large extent by the materials of construction and the concept design. The discussion is broken down into the materials used to construct the various concepts that passed the initial screening, since the various individual concepts (rigid, compliant, multiple, etc.) are, in many cases, constructed from two or more materials.

4.1.1 Steel Fabrication/Construction

Concepts 3 and 4 for OTEC cold water pipes are based on the use of steel. The two concepts are similar, except that Concept 4 has three or more articulated joints, which allows a thinner wall to be used. Both concepts have been studied to determine which grades of steel and which fabrication processes are most suitable. Approximate costs have also been determined for the several material variations, and risks in accepting cost data have been identified where applicable. Technical risks that could lead to degradation in properties are also identified.

The candidate steels and their mechanical properties are described subsection 3.1.3.1. Because a minimum tensile strength of the order of 100 ksi is required for Concept 3 per structural analysis, HY80 was

chosen. HY100 steel also meets this requirement, but the added cost and complexity of fabrication does not justify its use. ASME SA516 Grade 70 steel was selected for Concept 4 because the articulated joints allow thinner walls and lower stresses.

4.1.1.1 Fabrication Methods

Fabrication of the steel pipe is expected to be according to standard industrial procedures. Plates will be rolled and then welded by submerged arc or vertical MIG (Metal-Inert Gas) methods into 100-ft long cylindrical sections. Flanges will be machined and welded to the shell sections which will be assembled on-board. If the U.S. Coast Guard notch toughness requirements are not applicable, subsection 3.1.3.1, then single pass welding can be used. If the USCG requirements do apply to OTEC, however, multipass welding will be needed to ensure high toughness welds.

A welding technique that lends itself to the assembly of the segments during deployment is the "3 o'clock submerged arc" process, Figure 4.1-1. This technique is widely used for field fabrication of welded steel storage tanks; it allows submerged arc welding of horizontal joints. A mobile cage structure is suspended from the top of the cylinder and positioned over the joint. The cage contains the welding equipment operator.

Figure 4.1-1. 3 O'Clock Submerged Arc Welding

A continuous belt seals against the tank wall to contain the protective medium. The cage and welder tracks around the joint on driven rollers. For a steel CWP, approximately 30 to 35 passes are required to make a weld and to minimize the weld-time-per-joint; four crews (two welders each, one inside and one outside) would work on the joint simultaneously.

Since towing and keelhauling are not feasible for steel pipes of the anticipated size and weight, a through-hull deployment technique will be used. The pipe will be assembled during deployment either by the 3 o'clock welding technique or by bolting. For the spar, through-hull deployment will take place on an auxilliary barge; the CWP would then be positioned under the spar using augmented flotation and then attached.

Rough costs have been established for Concepts 3 and 4 based on conversations with American Bridge and Iron (Division of United States Steel). Fabrication costs per pound were estimated for the four candidate steels in terms of the following variations:

- In-shop fabrication
- Shore facility fabrication
- Single-pass welding
- Multiple-pass welding (high notch toughness)
- 3 O'clock submerged arc welding.

Raw material costs were obtained from U.S. Steel. The cost of corrosion protection was based upon using a 0.1-in. corrosion allowance and a cathodic protection system with a coating. The cathodic protection costs were roughly estimated using data generated by Lockheed for buoy systems and data provided by industrial consultants. A base of \$0.06/ft²/year was used resulting in a total of \$1,020,000.00 for a 30-ft diameter by 3000-ft long pipe with a 30-year life. The cost of coating was based on the use of coal tar epoxy and is estimated at \$1,140,000.00 (Reference 4-1). These costs per unit area were applied uniformly to all steel concepts even though it must be understood that significant variations in design and cost may result from the different CWP design approaches. This is especially true for the various articulated joint concepts.

The steel costs were divided into the following categories:

- The cost of a "basic" cylindrical shell; 30-ft diameter, 3000-ft long, by 1.00-in. wall thickness.

- The cost of a standard joint
 - flanges and bolts
 - 3 O'clock weld joint
 - articulated
- Stiffening rings.

The final costs were then based on the most recent wall thickness data, as determined through dynamic and structural analyses, and the number and types of joints.

Table 4.1-1 summarizes the basic shell cost data for the four candidate steels. Shipping costs are rough and, for convenience, the same amount per pound has been used for shipping either a finished product or raw material to the shore-site.

Table 4.1-1. Cost of Basic Cylindrical Shell, 1.00-in. Wall

STEEL		RAW MATERIAL COST (\$/LB)	FABRICATION COST (\$/LB)	SHIPPING COST (\$/LB)	TOTAL FOB SHORE (\$/LB)	TOTAL (4) BASIC PIPE (\$ X 10 ⁶)	TOTAL 0.1 IN. CORROSION ALLOWANCE (\$ X 10 ⁶)
A36	(1)	0.21	1.00	0.07	1.28	14.750	1.475
	(2)	0.21	1.25	0.07	1.53	17.62	1.762
SA516 GR70	(1)	0.30	1.00	0.07	1.37	15.790	1.579
	(2)	0.30	1.25	0.07	1.62	18.670	1.867
	(3)	0.30	2.25	0.07	2.62	30.190	3.019
HY80	(1)	0.61	1.33	0.07	2.81	23.160	2.316
	(2)	0.61	1.67	0.07	2.35	27.080	2.708
	(3)	0.61	2.67	0.07	3.35	38.600	3.860
HY100	(1)	0.64	1.40	0.07	2.10	24.200	2.420
	(2)	0.64	1.75	0.07	2.46	28.350	2.835
	(3)	0.64	2.75	0.07	3.46	39.970	3.997

- (1) FACTORY FABRICATION: NO NOTCH TOUGHNESS REQUIREMENTS.
 (2) ON-SHORE FABRICATION: NO NOTCH TOUGHNESS REQUIREMENTS.
 (3) ON-SHORE FABRICATION: USCG NOTCH TOUGHNESS REQUIREMENTS.
 (4) 1.00-IN. WALL X 30-FT DIAMETER X 3000-FT LONG.

Machined components, such as flanges, will have higher fabrication costs. Rough estimates based on ABI input are \$5.00/lb for A36 and SA516 and \$8.00/lb for HY80 and HY100. It is anticipated that the bolts will be specially made of a high-strength steel and the cost will be on the order of \$8.00/lb.

a. Concept 3 Costs. Three design variations are considered for Concept 3. The first is a cylindrical shell made up of 100-ft long segments with welded-on flanges; the segments are joined by bolts either on-shore or during deployment. The second is 100-ft long ring-stiffened segments that are joined either on-shore or during deployment by 3 O'clock submerged arc welding. The third is a built-up section, all-welded CWP featuring longitudinal stiffeners on the internal surface and ring stiffeners on the outside; this bolted flange concept requires machining the flange to provide the joining surface and bolt holes, and machining the bolts (studs) and nuts from high-strength steel (the leading candidate is 4330 V vanadium modified steel). However, the welded joint requires only stiffening rings which can be rolled and welded onto the cylindrical shell segments at a low cost.

b. Bolted Flange Joint. The basic wall thickness of the cylinder is 1.50 in. to which the corrosion allowance is added (0.1 in.). The flange joint cross-section is shown in Figure 4.1-2. The fasteners will be machined studs with hex head nuts and 0.125-in. thick washers on both ends. The stud diameter is 2.50 in. The costs are shown in Table 4.1-2. for pipes both with and without USCG notch toughness requirements. The major result of imposing notch toughness requirements is that multi-pass welding is necessary and this substantially increases fabrication costs. The no notch-toughness requirement pipe is estimated as costing \$50.06 million; the high-toughness pipe is estimated at \$67.24 million for a 2800-ft long pipe (spar). Both costs are based on a shore-facility fabrication which eliminates handling and shipping 30-ft diameter by 100-ft long sections to a shore-site.

c. 3 O'Clock Weld Joint. Again, the basic wall thickness is 1.50 inches. A 2.00 by 10.00-in. stiffening ring is welded at the midspan of each 100-ft long section. The cost of joining the sections by the 3 O'clock process is estimated at \$20,000 per joint and is to be performed

CONCEPT 3: BOLTED FLANGE

CONCEPT 4: CAPTURED FLANGE

Figure 4.1-2. Concepts 3 and 4

Table 4.1-2. Steel Cold Water Pipe Costs

CONCEPT	CYLINDRICAL SHELL COST*	FLANGE/BOLT JOINT COST	RING/LONG STIFFENER COST	ARTICULATED JOINT COST	COATING COST	CATHODE PROCESSOR COST	TOTAL COSTS
CONCEPT 3 (RIGID)							
BOLTED FLANGE (HY80)							
NO NOTCH REQUIREMENT	40.43	7.82	-	-	1.06	0.96	50.06
NOTCH REQUIREMENT	67.61	7.82	-	-	1.06	0.96	67.24
AN-WELDED (HY80)							
NO NOTCH REQUIREMENT	40.43	-	0.43	-	1.06	0.96	42.87
NOTCH REQUIREMENT	67.61	-	0.62	-	1.06	0.96	60.24
BUILT-UP SECTION (HY80)							
NO NOTCH REQUIREMENT	18.32	-	21.19	-	3.59	3.22	46.32
NOTCH REQUIREMENT	26.12	-	30.20	-	3.59	3.22	63.13
CONCEPT 4 (ARTICULATED)							
CAPTURED FLANGE (SA516 GR70)							
NO NOTCH REQUIREMENT	14.81	-	-	2.63	1.06	0.96	19.35
NOTCH REQUIREMENT	23.96	-	-	4.09	1.06	0.96	30.06
BIPLANAR FLEX JOINT (SA516 GR70)**							
NO NOTCH REQUIREMENT	14.46	-	0.30	7.36	1.06	0.96	24.26
NOTCH REQUIREMENT	23.39	-	0.48	11.89	1.06	0.96	37.77

* COSTS ARE IN MILLIONS OF DOLLARS FOR A 2800-FT LONG PIPE (PUERTO RICO SITE-SPAR)

**THREE JOINTS

as part of the deployment. The costs are shown in Table 4.1-2 and are estimated at \$42.87 million for a steel pipe with no notch-toughness requirements and \$60.24 million for the high-toughness pipe for a 2800-ft long pipe (spar).

d. Built-Up Section CWP. In this design, a thin wall (0.625-in. basic) can be used because the circumferential and longitudinal members add strength and stiffness. Figure 4.1-3 shows the cross sectional thicknesses for this concept after the 0.0-in. corrosion allowance has been added. Cost estimates were based on the per pound costs in Table 4.1-2, even though the increased amount of welds may increase the fabrication costs.

- **ADVANTAGES (OVER RING-STIFFENED SHELL APPROACH)**
 - THINNER BASIC WALL THICKNESS
 - LESS TOTAL STEEL, LOWER WEIGHT

- **DISADVANTAGES**
 - MORE WELDMENTS; HIGHER FABRICATION COST/LB
 - MORE COMPLEX JOINT AT 100 FT SEGMENTS
 - OVER 3 TIMES THE SURFACE AREA TO PROTECT FROM CORROSION
 - INTERIOR SURFACE OF LONGITUDINAL STIFFENERS MUST BE PROTECTED FROM CORROSION; COATINGS AND CATHODIC PROTECTION METHODS QUESTIONABLE

- **CONCLUSIONS**
BECAUSE OVERALL COST APPROXIMATELY THE SAME AS RING-STIFFENED CYLINDER AND RISK IS HIGHER, THIS APPROACH WAS DISCONTINUED

Figure 4.1-3. Rigid Wall, Fixed Joint: Built-Up Section Steel Pipe

The cost of the cathodic protection system was increased proportional to the exposed surface area of the pipe. The longitudinal stiffeners add 1.357 million square feet of surface area since seawater wets both the inside and outside of the semi-circular members.

The estimated costs are shown in Tabel 4.1-2. The total cost for the no-notch requirement is \$46.32 million compared to \$42.87 million for a ring-stiffened cylinder. The increase in cost is due to the increased cost of corrosion protection: \$6.81 million for the built-up concept compared to \$2.01 million for the ring stiffened cylinder.

e. Concept 4 Costs. In Concept 4, several articulated joints are equally spaced along the pipe length to provide compliance and to thus reduce the loads. Two basic types of articulated joints were evaluated: a joint every 60 to 100 feet that would provide a small rotation of the order of 1 to 1.6 deg; and three to five gimballled joints which would provide relatively large rotations of the order of 5 to 10 deg. The pipe wall thickness will be 0.75-in. plus the corrosion allowance. Since the status of the structural design and analyses for articulated joints are not as advanced as for rigid pipes, typical sections were assumed for costing. These rough costs will be modified as more accurate configuration/cross section data is obtained.

f. Captured Flange Joint. A conceptual joint for providing 1.64 deg of rotation is shown in Figure 4.1-2. The pipe segments are fitted with flanges that are captured by a segmented housing or outer ring. Neoprene bumper pads are provided to reduce impact load and to eliminate damage caused by metal-to-metal contact. The joint is roll-and-welded construction and employs no machining except for several bolt holes provided for joining the outer ring segments. Due to the occurrence of impact loads, the high notch-toughness requirements (multipass welding) are applied. It must be emphasized that all dimensions shown in the figure are estimated and no detailed analysis has been performed. Indeed, there is a question as to whether the joint will work at all. The stiffness/rotation/stress relationship is such that there is doubt over the feasibility of any small rotation joints for the CWP.

The rubber bumper pad cost was roughly based on neoprene bridge pad cost and was estimated as \$10.00 per pound installed.

The pipe costs are shown in Table 4.1-3. The total pipe cost is estimated as \$19.35 million for a 2800-ft long CWP (spar).

Table 4.1-3. Steel CWP Total Costs*

• RIGID WALL, FIXED JOINT: ALL-WELDED, RING STIFFENED						
PLATFORM	FABRICATION	SCREEN	HULL ATTACHMENT	DEPLOYMENT	NON-RECURRING	TOTAL
BARGE (2500-FT PIPE)	38.28	0.1	3.00	14.1	16.1	71.58
SPAR (2800-FT PIPE)	42.87	0.1	3.00	11.9	16.1	73.97

• RIGID WALL: ARTICULATED JOINT, RING STIFFENED						
PLATFORM	FABRICATION	SCREEN	HULL ATTACHMENT	DEPLOYMENT	NON-RECURRING	TOTAL
BARGE (2500-FT PIPE)	22.45	0.1	3.00	12.8	16.1	54.45
SPAR (2800-FT PIPE)	24.26	0.1	3.00	10.6	16.1	54.06

*IN MILLIONS OF DOLLARS; NO NOTCH TOUGHNESS REQUIREMENTS IMPOSED; SOCKET HULL ATTACHMENT.

g. Gimbal Joint. A conceptual joint for providing rotations of the order of 5 deg by use of three gimbals with pin/bearing support is shown in Figure 4.1-4. This joint is based on a GMDI biplanar flex concept for the hull-pipe support.

The structural member thickness has been roughly estimated to obtain costs. The cost of steel for the pivot/bearings is estimated as \$12.00/lb, because these components require machining and are fabricated from thick stock (up to 12 in.). The pin itself will be of special concern since the 12-in. diameter means that full mechanical properties cannot be obtained in standard bearing materials. The cost of the fabric-reinforced neoprene bellows was estimated as \$20.00/lb installed, which is roughly twice the cost of raw stock. The no notch-toughness requirement cost of the pipe is estimated as \$24.76 million of which \$7.35 million is associated with the three joints. The cost for the 2800-ft long pipe with the imposition of full notch-toughness requirements is \$37.77 million, with \$11.89 million associated with the three joints. The costs are shown in Table 4.1-3.

Figure 4.1-4. Concept 4 Gimbal

Other types of gimbal joints were also investigated. In one of the concepts which may be applicable, flexures replace the pin bearings, thus eliminating sliding/rotating parts. This design is illustrated in Figure 4.1-5. The cost is expected to be approximately the same as that calculated for the biplanar flex joint.

A centrally located "monoball" bearing joint was also investigated (subsection 3.3.4.3.b). Unlike concrete pipe concepts, however, the steel concepts do not have a central support, and all the loads must be transferred from the wall to the bearing joint and back out to the walls.

Figure 4.1-5. Alternate Gimbal Joint

The bearing support structure will therefore be substantial and may restrict water flow. There is also a question as to the structural integrity of this type of joint under a torsion load. Since no great cost differences are anticipated, this concept was dropped for steel pipes.

The total CWP costs were calculated for both the spar (Puerto Rico site, 2800-ft long pipe) and the barge (Brazil site, 2500-ft long pipe) and are shown in Table 4.1-2. A socket-type hull attachment was assumed for both vessels. Deployment and non-recurring cost estimates were based on on-shore fabrication and through-hull deployment. A screen cost of \$100,000 was used for all pipes. The articulated joint design is based on the use of three biplanar flex joints as described above.

4.1.1.2 Risks Associated With Steel Pipes

The use of a steel pipe has certain associated technical risks as well as the risks associated with estimating costs based on the conceptual designs which may not be valid for the final detail designs.

Common to both the Concept 3 and 4 pipes is the assumption that the cathodic protection costs per unit area will be the same for all steel designs considered, and that they will be based on costs for existing ocean structures. The size and lack of access of the CWP, as well as the complexity of articulated joint design, certainly makes this assumption risky. In addition, adequate shore fabrication costs are based on those of typical large steel welded structures which may vary from fabricator to fabricator. All costs will have to be adjusted during the preliminary design and the final detail design.

In addition to the above risks, there are also risks that are inherent to the specific pipe concept.

a. Concept 3 Risks.

Bolted Flange Joint. The greatest technical risk associated with this concept is the performance of the bolts for the projected life of the CWP. The use of high-strength steel at high load in the ocean environment will lead to stress corrosion cracking (SCC) problems. It is highly unlikely that bolts will last for the 30-year life and, since no practical way of periodic replacement exists, catastrophic failure can occur.

Reports that HY80 steel may exhibit lamination problems in thicknesses greater than approximately 1.50-in. are also of concern. These reports are unconfirmed, but, if true, may affect the weld quality and/or procedures.

b. 3 O'Clock Welded Joint. The risks associated with this concept are the assumptions that the method can be adapted to the CWP and that the cost per joint is realistic, especially for the at-sea deployment mode. The technique is widely used on land to fabricate large storage tanks, but the typical dimensions from the top of the section being welded to the weld is on the order of 10 ft. In the CWP case, this dimension is 100 ft. It is possible that the stiffening rings may have to be used to support the welding cage, which would require internal as well as

external rings. The technique must be developed to minimize the welding time and to provide procedures for placement and alignment of the segments. It is anticipated that each joint will be made using four two-man crews so that the high number of passes (30 to 35) may be made in a reasonable time (approximately two days). While longer times will not significantly impact the overall pipe costs, they will greatly increase risks of encountering storms or other hazards during deployment.

Another concern is the possibility that the welds would have to be stress relieved; the 1.50-in. thick HY80 steel is borderline for this requirement. If necessary, the operation can be performed on-board but it would require approximately two days per joint, which extends the deployment time.

c. Built-Up Section. The above risks associated with the use of steel pipes apply to the built-up section concept as well. This concept has two additional risks: cost uncertainty and performance uncertainty. The cost uncertainty arises from the assumption that the fabricated cost per pound is the same as for a ring-stiffened cylinder. The built-up section has a much higher number of welds since each longitudinal stiffener requires two full penetration welds 300-ft long, and welds to join the stiffeners at 100-ft intervals (the assumed length of the cylindrical segments). This will probably mean a higher per pound fabrication cost, possibly of the order of a \$0.50 per pound increase.

The performance uncertainty exists because the inside of the longitudinal stiffeners may not be protected by the cathodic protection system. Coating this area will be difficult, at best, and the coating will be virtually non-inspectable. Placement of anodes within the individual stiffeners presents a severe design challenge and it is anticipated that significant development will be required before a final design is reached.

d. Concept 4 Risks.

Captured Flange-Joint. There are four prime concerns with this concept. First and most important is the question as to whether the joint is even feasible: initial rough analyses of stiffness, rotations, and stresses indicate that the problem may be divergent and unresolvable.

Assuming that a design can be found, it must be noted that the thickness and cross sections used in the costing are rough estimates and may significantly differ from the final design; cost increases for the joints of up to 50 percent or more may occur. Furthermore, the 0.75-in. basic pipe wall thickness may be too thin, especially near the flange because the load is transferred eccentrically through only a portion of the pipe wall when the joint rotates.

The cathodic protection of the outer rings poses a problem. Sealed straps must be provided to maintain electrical continuity with the pipe segments and those straps must last 30 years, a severe requirement. The increase in cost of the cathodic protection system may be on the order of \$1 million because of this requirement and periodic maintenance may be needed. The compatibility of bonded rubber pads with cathodic protection requires study. Normally, bonded elastomers are not used with cathodic protection systems because of gas evolution at the metal-bond interface and subsequent blistering and debonding.

e. Gimbal Joint. The main risk associated with the use of gimbaled joints is the feasibility and reliability of bearings submerged in seawater for long periods of time without maintenance. To our knowledge, no bearings have been used under these conditions for more than six months. Submerged bearings are normally retrieved, lubricated, refurbished if necessary, and reinstalled at relatively short intervals (1 to 3 months). To compound the problem, the history obtained with submerged bearings has been for small bearings not approaching the size of that required for the CWP. Long-term lubrication is a problem which has not been addressed. The use of solid lubricants, non-metallic bearings, etc., introduces other problems which have not been addressed either. A significant program to design, develop, and test submerged bearings must therefore be performed prior to adopting this type of joint. The use of flexures in place of bearings may alleviate these concerns, although adequate corrosion protection is still a problem area. Again, a history of submerged-in-seawater usage is lacking and development testing is required.

As in the captured flange concept, cross sectional shapes and thicknesses were estimated and will have to be adjusted as analyses are performed. This will cause adjustments in the estimated costs.

The use of straps across the bearings will have to be supplied to provide electrical continuity for cathodic protection. These straps must also be protected or maintained to provide 30-year service; the average cathodic protection system life is 10 to 15 years.

4.1.1.3 Recommendations

Following are our recommendations:

1. Concept 3: Rigid Pipe

- For costing purposes, use the all welded pipe design (3 O'clock arc weld option). The segments are fabricated on-shore and assembled during deployment as described. The bolted flange concept is too risky because of the stress corrosion cracking problem associated with high-strength bolts; it is also high in cost. The built-up section design is higher in cost and has technical risks associated with the corrosion protection system. In addition, the estimated costs could be higher because of the extensive welding required to attach and join the longitudinal stiffeners.

- Use the costs based on the USCG no notch-toughness requirements. Add the imposition of the requirements as a risk factor with a known cost increment. Note that in all probability, the notch-toughness requirements will eventually be imposed.

2. Concept 4: Articulated Pipe

- For costing purposes, use the gimbal joint design because there is a real question as to feasibility of the captured flange concept.

- Assign a high risk factor due to the question of the feasibility of submerged bearings.

4.1.2 FRP CWP Fabrication/Construction

4.1.2.1 Fabrication Methods

Filament Winding. The logical fabrication process for the FRP CWP walls is filament winding. Filament winding consists of winding or wrapping continuous filaments, fiberglass roving in this case, on a mandrel (Figure 3.1-1). Because curved or cylindrical surfaces are ideal for this process, it is almost universally used for fabricating FRP pipes, tanks, and pressure vessels. For large FRP structures such as the CWP, the binder material is usually combined with filaments during the winding; the process is called wet winding. In this case the wet-wound CWP sections are cured on the mandrel by the appropriate resin/curing system; the resin will be either a polyester or vinylester, probably a high-elongation, premium grade.

The key advantages to the filament winding process are 1) the filaments (glass fibers) can be oriented in an extremely precise manner; 2) the resin content can be controlled very accurately; 3) the whole process can be repeated with a high degree of reproducibility independent of operator technique. This reproducibility feature lends the process to a relatively high degree of automation. Further, with the use of a collapsible mandrel (Figure 4.1-6), the fabrication cost per CWP section can be greatly reduced by re-use of the mandrel.

Pultrusion. The main feature of the pultrusion process is that it can produce very long or continuous FRP parts of constant cross section. The process forms these shapes by pulling glass fibers or reinforcements (mat, cloth, etc.) and an uncured (wet) thermosetting resin through a heated die where the resin is cured and the composite hardened. Almost any constant section shape is possible.

The reinforcement (usually a combination of continuous-strand glass roving, woven roving, fabrics, and surfacing veils) is fed through a resin bath to wet-out the fibers. Excess resin is then removed and the saturated reinforcement pulled through a die. The entrance section of the die is water-cooled to prevent premature curing. The remainder of the die is heated by a jacket through which hot oil circulates. Newer machines

Figure 4.1-6. Extraction of FRP Pipe Section From Mandrel

use radio frequency heating alone or in combination with oil heating to cure the resin. Polyurethane blocks, designed much like tank treads and shaped to the configuration of the pultrusion, then pull the finished shape from the die. (Figure 3.1-3 shows a typical pultrusion setup).

Constructions. Sandwich and hollow wall constructions can be made by the filament winding process. Core material for sandwich constructions, such as balsa wood and plastic foams, are usually laid up between winding wraps where the excess resin from the wet-out glass filaments acts as the adhesive to bond the FRP to the core. Pultrusions can also be used as core material; the laminate can be tailored so the axial fibers can be incorporated into the pultrusion, which lends itself readily to this kind of orientation. The filament winding wraps then become the circumferential or helical plies of the laminate. Pultrusion core construction is particularly well-suited to hollow or truss-like wall sections where added wall section stiffness is needed, but not the added weight resulting from the use of solid sections.

Core Materials. Other than the FRP pultrusions discussed above, the core materials to be used in FRP sandwich wall constructions are end-grain balsa wood and syntactic foam. The balsa wood will be used where permanent (30-year) buoyancy is not required and the syntactic foams where it is required. Both materials have been used extensively in marine structures for many years, and the syntactic foams have been used at ocean depths greater than 3500 feet. Table 4.1-4 shows the typical mechanical properties for these materials.

Table 4.1-4. Core Materials Properties

MATERIALS	DENSITY (LBS/IN. ³)	COMPRESSIVE STRENGTH (PSI)	COMPRESSIVE MODULUS (PSI)	SHEAR STRENGTH (PSI)	SHEAR MODULUS (PSI)
END GRAIN BALSA WOOD	6	750*	330,000*	158**	26,000
	11	1,910*	768,000*	298**	
	15.5	2,950*	1,164,000*	425**	
SYNTACTIC FOAMS	22	1,300	100,000	700	40,000
	26	1,500	112,000	750	42,500

*PARALLEL TO GRAIN

**NORMAL TO GRAIN

4.1.2.2 CWP Construction Methods

Rigid Wall Concept. The FRP cross section chosen as the best concept is the sandwich, or hollow wall, construction. The overall pipe concept then is a rigid, wall-supported pipe without articulating joints (Figure 4.1-7). The assembly will be accomplished by the use of bonded joints, as shown in Figure 4.1-8. The bonded joint was based on the standard FRP bonded joint (NBS Standard 15-69) used in water and sewer pipe and modified to our needs. In the standard bonded joint, the overwrap length is only 16t and the thickness, t, which is the pipewall thickness. We have increased the length relation to 20t and the overwrap thickness will be 1.25t.

Figure 4.1-7. Fiberglass Reinforced Plastic Pipe Concept: Rigid

From the analysis of the rigid FRP pipe concept, the solid, sandwich, and hollow-wall thicknesses are shown in Figure 4.1-9, with one exception: for the sandwich and hollow-wall pipes, the cross sections were modified to make the pipes neutrally buoyant to reduce the loads on the pipe for keelhaul deployment. Of the sandwich/hollow-wall pipe sections shown, the hollow pultrusion wall configuration is the most efficient and the least costly. Since it will be necessary to either seal off the hollow chambers to make this pipe construction neutrally buoyant for deployment, or provide buoyancy collars along the length of the pipe, some development will be necessary to determine a practical way to unseal the hollow passageways during or after the pipe is upended in the water, or to remove the collars after deployment. If this is not done, the high hydrostatic pressure at depth could collapse the walls in the sealed, hollow-wall method.

Figure 4.1-8. Wall Bonded Joint Configurations

Figure 4.1-9. Candidate FRP Wall Configurations

Circumferential ring stiffeners will be provided along the whole length of the pipe. The ring cross section will be as shown in Figure 4.1-10. The spacing of these rings will be every 60 ft of pipe length. While this spacing is much closer than is required from the structural analysis, it was chosen because it will result in a spacer for each fabricated section. The stiffener will then provide stability for each section and ease the problem of extraction from the mandrel and handling and transportation. The wall construction of choice, then, is the balsa wood core sandwich.

Figure 4.1-10. FRP Stiffener Cross Section

The pipe sections for this concept will be fabricated as follows:

1. The inner wall ply will be wet-wound onto a mandrel.
2. The end-grain balsa wood core will be laid up over this inner wrap. The balsa core is commercially available in panels backed with fiberglass mats with a finish compatible with the impregnating resin used for the FRP walls.
3. The outer ply of FRP, as well as the integrally wound stiffening ring, is then wet-wound over the core, and the resin is allowed to cure.

4. The cured, rigid composite of FRP skins and balsa wood core is then extracted from the mandrel (Figure 4.1-6). The extraction process is aided by collapsing the mandrel internally.

Sixty-foot pipe sections were chosen as the basic construction unit for this concept because they represent the maximum length compatible with existing mandrel and extraction technology and cost-effective fabrication processes. A maximum pipe section length is desirable because it minimizes the number of joints needed to achieve the required CWP length.

The extracted 60-ft pipe section will then be brought to the assembly area on cradles and fitted up (butted) horizontally to another pipe section, Figure 4.1-11. The inner and outer joint faying surfaces will first be prepared by sandblasting and solvent-wiping. The outer joint will first be wet-wrapped with a highly unidirectional glass roving mat of approximately 98 percent by weight fill direction glass fiber content. The inner joint will then be laid up using the same unidirectional roving mat. Here the wet-out mat will be allowed to advance in cure to high tack consistency, which will facilitate its adhesion to the inner pipe walls.

Figure 4.1-11. Assembly and Bonding on FRP CWP Section On Shore

The joint lay-up will then be pressed against the inner pipe walls by means of a vacuum or pressure bag backed with a structural "spider," and allowed to cure. The joint configuration will be as previously shown. The circumferential wrapping of the joint with the unidirectional roving will provide high glass fiber loading in the pipe's axial direction. Since the circumferential pipe loads are fully taken care of by the pipe walls, the use of a high-axial glass content overwrap will provide an added factor of safety in the joint for carrying the axial loads.

Modified Stockade Wall Concept. One nuance of the FRP sandwich, or hollow-wall, concept, the stockade wall, was analyzed and resulted in the following CWP. The idea is based on the exclusive use of pultrusions to fabricate the pipe, and combines fabrication and deployment. The concept was originally proposed by Goldsworthy Engineering of Torrance, California, and used pultruded FRP sections that can be fitted together by tongue-and-grooves, which are simultaneously bonded with a rapid-cure resin. Goldsworthy originally helically wrapped these sections to form fuel tanks; we have modified the concept to the following combined fabrication and deployment scheme:

Step 1. Begin with a helically wound, pultruded FRP inner pipe wall fabricated by the standard tank-making method. These would be tongue and grooved and adhesively bonded.

Step 2. Lay-up onto this cylinder longitudinal pultruded FRP sections that are butted together and serve as the axial tension members of the pipe. These longitudinal staves will also be simultaneously bonded to the inner wrap by means of a quick-curing resin.

Step 3. An outer tongue-and-grooved FRP pultrusion will then be helically wrapped around this composite and, again, simultaneously bonded by means of a quick-setting resin to both the longitudinal pultrusions and itself. The whole pipe is continuously fabricated and deployed in this manner (Figure 4.1-12).

The cross sections of the core and inner and outer wrap pultrusions are shown in Figure 4.1-13. The core longitudinal hollow sections will be allowed to fill with water while the inner and outer pultrusions will be filled with 17 lb/ft³ balsa wood co-pultruded with the FRP during the pultrusion's manufacture.

Figure 4.1-12. Combined Helical and Longitudinal CWP Fabrication and Deployment Scheme

Figure 4.1-13. FRP Stockade Pipe Cross Sections

Multiple FRP Pipe Concept. The FRP multiple pipe concept for the CWP is based on the use of commercially available pipe which need not be custom-fabricated in order to meet the structural requirements. In contrast to the polyethylene multiple-pipe concept where cables take the ballast loads, the FRP pipe walls will carry the ballast loads.

The FRP pipes are no longer off-the-shelf because of the special fiberglass orientations needed (such as 0-deg direction fibers). Furthermore, the off-the-shelf mechanical joints available in this type of pipe (bell and spigot) are inadequate for the design loads; bonded joints will have to be used, and this automatically favors on-shore fabrication and keelhaul deployment. Keelhauling then favors neutral buoyancy which entails a sandwich construction; this, in turn, leads to a balsawood core sandwich which is not an off-the-shelf pipe.

The largest commercially available off-the-shelf FRP pipe is 12-ft diameter, 30-ft long. (Larger pipe diameters are not transportable by common carriers.) On the basis of this and the analysis of the power losses versus number of pipes, the FRP multiple-pipe CWP will consist of six 12-ft diameter pipes. As stated above, the pipe wall will be sandwich-constructed of 1-in. filament-wound FRP divided equally on both sides of a 17 lb/ft³, 1.10-in. balsa wood core (the same as shown in Figure 4.1-9, except with different dimensions). Circumferential ring stiffeners will be wound into the outer wall at proper intervals, if necessary (as in the 30-ft diameter rigid wall FRP pipe in Figure 4.1-7). Each pipe will support the ballast weight loads through the wall and a plenum chamber/adaptor will collect the flow of the pipes at the top.

Again, a bonded FRP joint will be used with the same configuration as used in the 30-ft pipes (see Figure 4.1-8). The assembly scheme will also be the same (Figure 4.1-11), except that six pipes of the required length will be fabricated instead of one. The number of bonded joints for each pipe will be doubled because of the 30-ft pipe section unit length.

Bottom-Mounted FRP Concept. The bottom-mounted pipe will be fabricated and constructed in exactly the same manner as the 30-ft rigid sandwich wall concept, with the following exceptions:

1. The FRP pipe will only be 2600-ft long.
2. The core material will be syntactic foam of two densities: 22 lb/ft³ and 26 lb/ft³.
3. The 22 lb/ft³ foam will be used down to 1500 ft of water depth and the 26 lb/ft³ will be used for the remainder of the pipe.
4. The 22 lb/ft³ foam will be used for the buoyancy section and will be incorporated as part of the sandwich wall in the upper 300 feet of the pipe. The positive buoyancy will be 1.5×10^6 lb.
5. The core thicknesses for the remainder of the pipe is the thickness required to achieve neutral buoyancy in order to aid keelhaul deployment. The FRP wall thickness is 1-in. divided equally on each side of the core. Ring stiffeners will be placed at 60-ft intervals, as with the FRP sandwich-wall pipe, using the configuration shown in Figure 4.1-10. No stiffeners will be required in the buoyancy section.
6. The core thickness is 1.25 in., for the 22-lb/ft³ sections and 1.38 in. for the 26-lb/ft³ sections.
7. The upper 300 ft of pipe or buoyancy section will use 16.8 in. of foam for the total core, with the same FRP face sheet thicknesses used in the rest of the pipe.
8. Assembly will be by bonded joints (see Figure 4.1-8). The same fabrication and assembly scenario will be used as in the sandwich FRP 30-ft pipe. The thicker buoyancy section transition joint to the thinner sandwich of the lower pipe will require a special outer wall joint wrap which has not been detailed.

4.1.3 Concrete CWP Fabrication/Construction

Construction afloat and construction on-shore are the two most promising methods for the OTEC CWP. On-shore construction consists of precasting pipe sections at a coastal site; construction afloat uses slipforming at the deployment site or in protected waterways of acceptable depths.

The following sections discuss both of these fabrication/construction techniques as they apply to a concrete OTEC CWP. The methods, which are general in nature, apply to all concrete CWP configurations discussed in subsection 3.3.3.3, and were assessed by C. Richardson and others (Reference 3-22). The selected method, on-shore construction, was used as the basis for assessing the fabrication cost of a concrete CWP.

4.1.3.1 Construction Afloat

By this method, the structure is built up progressively while supported by the water's buoyancy. A protected, accessible construction site must be chosen with water deep enough to make this construction technique economically viable. This method also has the advantage of not requiring the heavy, revolving crane necessary for handling precast sections.

Construction would be completed by the slipforming method which is practical and would be economical for a continuous concrete CWP with long lengths between articulated joint sections. The CWP base is constructed onshore and floated to the selected construction site. Then slipforms are installed and the concrete walls raised while the CWP slowly sinks. Concrete would be supplied from a construction barge equipped with a batch plant; the batch plant and construction barge would be sized to supply the daily demands for concrete.

CWP neutral buoyancy is controlled during construction by ballasting the pipe internally. The CWP can always have positive buoyancy with seawater added to make it sink and preserve the stability of the operation. The positive buoyancy is also required to maintain the slipforms and

concreting positions within a reasonable working distance above the seawater line. Inlet seals and/or internal collapsable diaphragm would be required to control the buoyancy.

The slipforming operation will not alleviate the requirement for joints within the CWP. Weather conditions and longitudinal prestressing (i.e., effective lengths for minimum prestress loss) require breaks in the construction operation. The joint lengths would be established from a combination of structural requirements and those of the prestressing operation. The lengths of these sections could be up to 150 feet to suit the prestressing operation.

The CWP walls have to be thick enough to resist the differential external hydrostatic pressures and current forces on the exposed fabricated pipe sections. The impact of this requirement for the CWP concept evaluation was not investigated. The concrete pouring schedule must be coordinated with the sinking rate to allow a sufficient setting and curing period prior to entering the water to ensure adequate strength and impermeability and to prevent wave erosion.

Slipforming a CWP 3000-ft long is estimated to take about one year to complete. Construction would be performed parallel with the schedule of the platform hull. The CWP would then have to be positioned below the platform and fitted into its final location. This CWP deployment scenario is discussed in greater detail for CWP concepts for the spar platform.

The details of the slipforming method depend on the final concrete mix design and CWP configuration. The program schedule did not allow the development of a complete, detailed schedule and construction sequence for the slipforming method. However, the following conclusions can be made: CWP lengths exceed the depths of sheltered waters near the proposed OTEC platform deployment sites (Brazil and Puerto Rico) and the relatively thin wall sections and high axial prestress will require development to guarantee the quality of the cast workmanship, or what would be anticipated as being limiting tolerances on wall thickness and

circularity requirements for the CWP. The first conclusion requires construction at the deployment site; the second requires a test program to completely develop the construction sequence. These requirements for CWP fabrication result in high risk item CWP concept evaluation. Therefore, the slipforming method was not considered in greater detail during this phase of the CWP concept evaluation.

4.1.3.2 Construction On-shore

The on-shore construction method was used as the basis for assessing the fabrication/construction costs that would be involved in producing the post-tensioned concrete CWP. A coastal site was used to estimate costs; the appropriate site layout (proposed in Reference 3-33) is shown in Figure 4.1-14.

The CWP would be constructed in nominal 25-ft sections, jointed together in 50-ft sections by post-tensioning. The completed sections would be ready for shipment and deployment. The 50-ft lengths are appropriate from the standpoint of on-site handling with any existing form of lifting devices (e.g., gantry cranes or tractor crawlers), and of transportation by barge. Because the distances between the fabrication and the deployment sites will probably be large, and transit times correspondingly long, it will be necessary to use a number of barges to meet the fastest possible erection times at the deployment site. Equipment modules transported for offshore platforms are 35- to 50-ft high and have weights comparable to that of the concrete pipe sections. The choice of 50-ft section lengths for precast pipe lengths is therefore within current technology and does not represent a high-risk item for the concrete CWP.

The daily concrete quantities required for CWP fabrication are small so that the concrete could be supplied by ready-mix trucks. This assumption avoids the cost of setting up a concrete batch plant, as well as storage facilities for aggregates and cement. Protection of the aggregate is required for production and placement of high quality concrete required for the CWP. Contamination and deterioration of aggregate quality can arise from improper storage. Stored on the ground or on a slab at grade,

the aggregate can become contaminated by dirt, ice, and undersized particles. Cement must be stored and used in such a manner that it does not age excessively. It must also be completely protected from moisture; a tropical or humid atmosphere may present difficulties in this regard and may require dehumidification of the storage shed.

The site area required for concrete CWP fabrication, and to some extent the method of handling completed pipe sections, depends on the number of units to be stored. This, in turn, is linked to the towing distance to the deployment site and the speed of assembly at the site. To simplify the present study, it was assumed that storage space for all pipe will be required.

Several methods of handling and transporting the pipe sections was considered and costed. A gantry crane with a 140-ton capacity, 80-ft clear hook height, and 60-ft track spacing was selected as the basis for estimating the CWP fabrication costs. This crane was estimated to cost \$1.2 million dollars; the selected configuration, shown in Figure 4.1-14, appears optimum for handling concrete pipe sections. The alternative of using heavy mobile cranes to handle half sections in the assembly area and a gantry crane spanning a single section and traveling only over the assembly area was also considered. It was concluded that the larger gantry would be less expensive overall because past experience shows that mobile cranes are slower in operation and require frequent maintenance for both machinery and the traveling area. Furthermore, if two sets of CWP's, or other, similar pipes are required, additional unit costs due to purchase of the crane would be reduced.

The most economical means of transporting completed pipe sections is by crawler transporters which the offshore oil industry has used extensively for handling very large pieces of fabricated steel sections for steel jacket structures. The transporters would enable the use of a compact storage area and eliminate the need for any specially prepared areas.

Precasting the CWP requires approximately six months and two complete sets of forms (two upper and lower half sections) for efficient use of the facilities. The forms would be constructed as a built-up steel frame units; the panel units would be designed for self-stripping and equipped with access doors for concreting at hard to reach places.

The detailed fabrication sequence was based on a period of approximately 6 months; a 12-hour per day, 8-day detailed program of the daily tasks required for producing the units is presented in Figure 4.1-15. The sequence was scheduled to produce one-half pipe section (25-ft nominal height) per day. This is a 5-day turn-around on each pipe section formwork.

The reinforcing and stressing ducts would be placed around special formers. The bottom ring steel for the lower half unit is first formed, complete with all bent ducts and anchorages, on a special former then lifted over the full reinforcement cage former for the lower half unit. The remaining reinforcement and stressing ducts would then be added. Some rebar welding would be required at various bar intersections to provide sufficient rigidity to enable the cage to be handled without becoming distorted.

The reinforcement cage for the upper half unit is similarly erected on its special former. Only one former would be required to meet the casting sequence requirements.

Access for fixing reinforcement to the formers is provided by means of a mobile telescopic platform which acts as the crane for up-ending bundles of reinforcing bars and prestressing ducts. Two days are required for rebar assembly.

The remaining sequence of operations has been established to ensure the production of one-half pipe section per day. Concreting is performed via a mobile pump and would be completed in half shifts. Concrete placement may become difficult at congested areas of reinforcing bars and prestressing ducts (e.g., at the joints of the articulated wall joints),

KEY:

BED A

BED B

BED C

BED D

⊗ INDICATES OPERATION INVOLVING 140 TON CAPACITY GANTRY CRANE.

SEQUENCE OF OPERATIONS TO ACHIEVE ONE HALF UNIT FROM THE CASTING YARD EACH DAY.

Figure 4.1-15. Precast CWP Fabrication/Construction Sequence

and at areas of minimum section thicknesses. Good placement practices and quality assurance and inspection procedures are required to ensure proper placement of the reinforcement and compaction of the concrete in these areas.

The units are then heat-cured by covering and injecting low-temperature, low-pressure steam. Immediately after pouring, fresh concrete surfaces that are exposed to hot sun or drying winds may lose so much water that it sets and shrinks, even if the body of concrete is still plastic. Concrete curing must provide sufficient moisture to allow the complete chemical reactions necessary to produce strong, durable concrete. Some of these reactions occur rapidly, others require a much longer period. The exact time required is highly temperature-dependent, with different reactions responding differently to temperature/variations. The curing time chosen enables all desired reactions to occur.

The steam curing operation of the CWP is very important for concrete structures that are exposed to the ocean environment. Properly applied, low-pressure steam will improve the quality and durability of the concrete. Steam curing also improves concrete's resistance to sulphate attack and abrasion.

The pipe sections would be heat-cured for approximately 18 hours before beginning to strike forms (approximately 20 hours after placement). The pipe sections would be transported by the gantry crane to the assembly area when approximately 48 hours old.

At the assembly area, the lower half sections are placed on four support pedestals. The pedestal height is selected to allow the crawler transporters access under the units. The upper half unit is placed over the lower half unit; it is located by means of dowels within the wall section, the joint having been prepared with a thin epoxy mortar bed. The two half units are then temporarily stressed together ready for transportation to the storage area where the final stressing and grouting is performed. All prestressing tendons will be solid grouted within the stressing ducts.

The crawler transporters are also used to load the completed pipe sections onto deployment barges. The barges are equipped with acceptable jacking facilities to enable the transporters to be removed and the pipe sections lowered to suitable supports for sea transportation fastening.

4.1.4 Polyethylene CWP Fabrication/Construction

4.1.4.1 Multiple Pipe Concept

The polyethylene pipe to be used in fabricating the multiple pipe concept is made by extruding from a melt. The pipe will be obtained in 60-ft long, 5-ft diameter sections from the manufacturer. The 60-ft sections will be fusion-welded together into the desired CWP pipe length (2800 ft for the barge, 2500 ft for the spar). The fusion welding will be accomplished by means of a special machine leased from the pipe manufacturer. The weld schedule (time, pressure, and temperature) used will also be obtained from the manufacturer and will be based on the particular outside diameter and wall thickness of the pipe. In the actual design, the wall thickness will probably vary along the length of the pipe and it will be necessary to vary the weld schedule to match the pipe wall thickness. Fifty full length pipes will be required.

4.1.4.2 Stockade Wall Concept

For the stockade wall concept, the individual wall sections will be manufactured by extruding. The pipe wall will then be made up of full pipe length extrusions joined together by means of tongue-and-grooves (see Figure 4.1-16) into a 30-ft diameter pipe. The pipe will then be tied with turnbuckled fiberglass bands placed circumferentially around the outside diameter of the pipe at 25-ft intervals. If required, the same bands will be used on the inside of the pipe opposite the outer bands.

Welding these sections was rejected for the following reasons:

1. The thickest state-of-the-art fusion machine welds for polyethylene currently used in industry is approximately 2 in. The web thickness for this design is 3 to 4 in.

2. Hand welding is not reliable.
3. If a fusion machine weld were used, two means of welding the complete pipe length for each extruded wall section would have to be developed.
4. The fatigue properties shown in Figure 3.1-11 are based on parent material. No data is available on the fatigue endurance limit of the polyethylene in the heat affected zone.

Figure 4.1-16. Polyethylene Stockade Concept

4.1.5 Reinforced Elastomer CWP Fabrication/Construction

Two reinforced elastomer concepts have been studied:

- the Goodyear concept, which is a unidirectional steel cable reinforced neoprene steel ring-stiffened compliant pipe, and
- the Bell concept, which is a nylon fabric reinforced nitrite-PVD, pressurized annular wall pipe.

4.1.5.1 Goodyear Concept

The compliant wall membrane for the pipe will be made from 6-ft wide standard conveyor belting modified by the use of a 1/4-in. thick neoprene matrix on both sides of the longitudinal steel reinforcing cables. The present design uses 1/2-in. diameter cables spaced on 3-in. centers. These belts are obtained from the manufacturer in continuous lengths and will be joined longitudinally in 50-ft long tubular sections by means of vulcanizing at the manufacturer's so the 50-ft sections will have reinforcing cables in the pipe's axial direction (Figure 4.1-17). The cable ends will terminate into circumferential steel rings by means of bolted swagged eyes, or shackles, or captured swagged balls. To the outer pipe wall membrane, circumferential stiffener tubes will be attached with through-bolts attached to internal steel backing plates. The circumferential stiffener tubes will be placed at 6-ft intervals along the whole length of the pipe. In the present design, the stiffener tubes have a 7.25-in. outer diameter with a 0.25-in. wall at the top of the CWP and will vary at cross section intervals down the length of the pipe to a 5.25-in. outer diameter, 0.188-in. wall tube at the bottom of the pipe.

Figure 4.1-17. Cable Reinforced Elastomer Configuration

The 50-ft sections will then be joined by welding or bolting into a CWP of the desired length. At the present state of design, the 50-ft ring-stiffened sections will be joined into 100-ft sections on shore and final-assembled at sea to the full CWP length as shown in Figure 4.1-18.

Figure 4.1-18. CWP Deployment

The main fabrication concern with this concept is in the area where joints have to be made. Ideally, the pipe would be made in one piece; that is, the reinforced elastomer membrane would be a tube 30 or so feet in diameter to which the stiffener rings, hull joint, bottom weight, and screen are attached. In the present state of this concept, the pipe is divided into sixty sections of 50-ft lengths. The joints then require termination and attachment of the cable ends into the joint rings.

Exposure of the cable ends and the joining ring attachment to seawater now presents a corrosion problem and the concomitant problem of protection. Stainless steels, austenitic types, are particularly vulnerable in this application because of their susceptibility to crevice corrosion. Carbon steels suffer from general corrosion and if the joining rings are a corrosion-resistant alloy such as one of the super alloys, galvanic corrosion will result. Further, the rubber-covered cables are more noble than the exposed joints, and this also results in galvanic corrosion.

Impressed current cathodic protection is unsatisfactory because hydrogen can be generated during the protection process at the rubber/steel interface and cause debonding.

Sacrificial anodes present a problem of life and added weight for the pipe. The anodes will also lose weight over their life, and this must be taken into consideration in the dynamics and structural analysis of the pipe. A passive protection, such as a galvanizing or coating with an organic material, is the only other way the joints can be protected. Galvanizing cannot achieve the 30-year life required and must be renewed periodically. Of the organic coatings, neoprene is the only one that has a 30-year life expectancy on steel in seawater (see Reference 4-2), however, the coating reliability depends on surface preparation of the steel and quality control exercised during the coating application.

4.1.5.2 Bell CWP Concept

For this concept, the compliant wall membrane will be made from nylon fabric reinforced neoprene elastomer. The annual sections will be built up by means of the joint shown in Figure 4.1-19. These 30-ft inside diameter annual sections will then be joined together by means of circumferential steel annual coupling rings which allow the pressurizing water to flow freely between sections. The sections will then be joined together into the final CWP length, either on-shore or at-sea, by means of these coupling rings.

Figure 4.1-19. OTEC CWP Candidate Joint

4.2 CWP INSTALLATION AND AT-SEA DEPLOYMENT PROCEDURE

Installing or connecting the CWP to the platform will be the final stage of the deployment of the OTEC system before plant operations begin. Because of the length of the CWP, this installation procedure will have to be done at sea in water deep enough for the pipe to be in the vertical position. The drag of the CWP when connected limits platform transiting to the operating site to less than 2 knots; moving is therefore a long, expensive process. Because of this, the CWP will be connected to the platform at the operating site. This concept was modified at the end of this task to consider connecting the grazing plant CWP off the coast of Brazil and not take into account that the proposed operating site is a long distance away. The scenario was written for connecting the CWP at the operating site, but a comparison chart is provided to show the costs associated with the two alternate connection sites for the Brazil plant.

This section discusses all at-sea processes required to connect the CWP to the platform. It includes moving the CWP from the shore to the platform, supporting the pipe during transit, and final connection to the platform.

4.2.1 Approach

Because of the inherent differences between the barge concept and the stationary spar platform, preliminary connection procedures were developed for each concept. With these procedures outlined, at-sea deployment scenarios were developed for each site, as required.

Three scenarios were developed for CWP-platform deployment and connection that were dependent on the pipe's fabrication location:

- On-shore. The CWP is assembled into one continuous length and towed to the platform.
- On-shore. The CWP is barged in sections to the platform and then assembled.
- At-sea. The CWP is fabricated at the platform from raw material that is barged to the site.

The third scenario was discarded during preliminary screening because of the complexity involved and the time required for at-sea fabrication. At-sea operations are also subject to delays caused by platform motions not encountered on land; on-shore fabrications are much faster from every standpoint.

Deployment schedules were developed from the selected scenarios and were plotted on a time schedule to show the time required for each deployment phase. Different schedules were prepared for each scenario because of the differences between platforms and operating sites. For all schedules the equipment and vessels were mobilized from the gulf coast states even though this is an additional cost burden on the grazing platform because mobilization to Brazil is a longer transit. Nevertheless, it was felt that the gulf states area was the closest area where the vessels required could be obtained in sufficient numbers and with any degree of certainty.

Analyses correlating CWP concepts to scenario parameters were performed, and the specialized equipment required for each CWP concept scenario was determined to limit each CWP to one scenario. Costs were then calculated and tabulated for each schedule and scenario in terms of site and concept.

4.2.2 Results

The approach used to determine a procedure for each type of CWP and site was aimed at minimizing risk, both in terms of at-sea deployment taking longer than scheduled, and the risk to the CWP itself. Positive control of the CWP during all phases is necessary: minimum time at sea decreases downtime risk caused by inclement weather; handling CWP sections during at-sea assembly increases risk; and working below the water's surface both increases risk and decreases efficiency.

The favored scenario, therefore, is that in which the CWP's are assembled on-shore and towed to the site. This scenario was used unless costs or complexity indicated that such one-piece deployment was less feasible.

Summaries of the scenario schedules developed and ROM costs generated are presented below for each scenario. Detailed costs and time schedules are presented in Appendix H.

4.2.2.1 Connection Procedures

Connection procedures for the spar platform differ from the barge procedure because of the small central column in the spar: this column does not allow the CWP to be deployed from the spar. All CWP's for the stationary platform must therefore be deployed from a construction barge. Keelhauling and connecting the CWP from a vertical position beneath the platform is necessary for all connections to the spar.

The procedure for keelhauling the CWP from a vertical position beside the platform for the final connection to the transition is one of the last steps in Scenario 1. This procedure would be required for all CWP connections to the spar platform. For the spar, the process must be done remotely or with divers working at a 200-ft depth. No other option appears feasible at this time.

The connection procedure for the barge platform will be a function of the deployment procedure. The open center well on the barge allows deployment of Scenario 2 CWP's through the well. The Scenario 1 CWP would be keelhauled under the barge and connected; this connection can be done underwater or the end of the CWP can be raised up through the well, clear of the water, for the connection to be made.

4.2.2.2 Scenario 1: Light CWP's

In Scenario 1 the CWP is assembled to its full length at a shore site. Once assembled and afloat, it is towed to the deployment site, up-ended to a vertical position, and keelhauled under the platform for final connection.

Investigation and analysis of this procedure showed that it was most applicable to a CWP with a density close to that of salt water. If the pipe's density is not close to that of salt water, there will be excessive stress developed in the CWP using this procedure. The addition of

flotation collars or controlled flooding which might allow this scenario to be used for dense material CWP's is considered too costly and complex. Another problem can develop during this scenario if the lowering vessels do not maintain proper separation: the CWP will kink. Positive control at all times during this type of scenario is necessary. Additional analysis will be required to ensure that no excessive stresses are introduced during the lowering procedure due to the mass of water in the CWP.

Towing long pipes on the water's surface is feasible. Favorable weather conditions must be chosen for the tow, and extra tugs and picket boats should stand by not only to assist the towing vessels, if necessary, but also to warn other vessels off.

Preliminary calculations show that even in a 44-ft wave with a 13.8-sec period, the stress level will only be about 1.9 ksi in an FRP pipe. Also, there is ample power in the work boats and tugs to tow the CWP. Temporary end caps would be used to keep the CWP floating as shallow in the water as possible; they would be removed when the up-ending procedure begins to allow the pipe to flood.

Any of the CWP's that are to be deployed by this method will require a large stabilizing weight to keep them in an acceptable vertical orientation during currents. Analysis indicates this weight will be on the order of 2 million pounds. Because lowering 2 million pounds to the depth of the CWP's requires extremely large equipment, a freeflooding steel container capable of holding the appropriate amount of cement will be lowered. When the CWP reaches the vertical position and is connected to the platform, a cementing unit on the lowering vehicle will pump cement into the container to displace the water and provide the desired stabilizing weight.

A step-by-step procedure for Scenario 1 for the Brazil grazing platform is given in Table 4.2-1. Figures 4.2-1 through 4.2-5 show the major sequence of events during the performance of Scenario 1. The number and size of craft shown in Figure 4.2-2 may be altered when detail analysis of this scenario is developed, but those shown are considered a good estimate for the Brazil tow and deployment. For the Puerto Rico stationary platform, this scenario would vary only in the length of time for

Table 4.2-1. Scenario 1 Procedure

1. MOBILIZE TUGS, BARGES, AND WORK BOATS AT NEW ORLEANS
2. EMBARK TUGS, BARGES, AND WORK BOATS FROM NEW ORLEANS TO SALVADOR, BRAZIL, SHORE ASSEMBLY SITE
3. MAKE UP TOW, INCLUDING PROVIDING SPECIAL TOWING TACKLE, PICKET BOATS AND WORK BOATS
4. PIPE IS MADE UP INTO 2500-FT LENGTH PROGRESSIVELY; PIPE SECTIONS ARE FED THROUGH PIPE-JOINING EQUIPMENT AND THEN FED INTO LAGOON OR OTHER PROTECTED WATER AREA
5. SPECIAL HARNESS IS SECURED TO HEAD END OF PIPE ASSEMBLY
6. TUG PICKS UP SPECIAL HARNESS AND TOWS PIPE OUT FROM SHORE AS PIPE IS BEING MADE UP PROGRESSIVELY ON SHORE
7. PICKET BOATS PATROL ALONG THE PIPE IN THE WATER TO PREVENT OTHER CRAFT FROM DAMAGING THE PIPE
8. AS THE PIPE IS PROGRESSIVELY MADE UP AND FED INTO THE WATER, SECONDARY HARNESS ASSEMBLIES ARE INSTALLED AT THREE LOCATIONS ALONG THE PIPE'S LENGTH
9. IF REQUIRED, DEPENDING ON THE PIPE DESIGN, THE STABILIZING CABLE IS PROGRESSIVELY FED INTO THE PIPE AS EACH SECTION IS ADDED
10. AFTER THE LAST PIPE SECTION HAS BEEN MADE UP TO THE 2500-FT LENGTH, THE BITTER END OF THE STABILIZING CABLE IS PICKED UP AND SECURED TO ONE OF THE ASSIST TUGS
11. THE LEAD-END OF THE SPECIAL HARNESS SECURED TO THE HEAD END OF THE PIPE ASSEMBLY IS SECURED TO THE TOW TUG
12. THE THREE TUGS ARE MOVED TO THE SECONDARY HARNESSES WHERE EACH SECURES A HARNESS TO AN AFT BOLLARD (BLACK TO ASSIST AS REQUIRED)
13. THE PICKET BOATS ARE ALERTED AND READIED FOR THE TOW
14. WEATHER PREDICTION SERVICE IS CONTACTED AND AT THE APPROPRIATE "WINDOW" TIME, THE TOW BEGINS
15. A FLOODABLE TANK IS FABRICATED TO ACT AS THE STABILIZING WEIGHT WHEN FILLED WITH CEMENT

Table 4.2-1. Scenario 1 Procedure (Concluded)

16. TANK, CEMENT PUMPING EQUIPMENT, AND CEMENT MIXES ARE LOADED ON A BARGE
17. COMPLETE TOW OF CWP AND ALL EQUIPMENT TO THE DEPLOYMENT SITE
18. BEGIN UP-ENDING PROCEDURE FOR THE CWP FROM HORIZONTAL TOW MODE TO VERTICAL OPERATIONAL MODE
19. COMPLETE TOW OF THE 100-TON CRANE BARGE TO THE SITE
20. SECURE LINE AND HARNESS TO THE STABILIZING TANK
21. MOUNT CONCRETE HOSE INTO THE TANK WITH A QUICK DISCONNECT ATTACHMENT
22. WITH THE CRANE BARGE, LIFT AND LOWER TANK TO BOTTOM AT CONTROLLED SPEED WITH CWP FOLLOWING. COMPLETE THE UP-ENDING PROCEDURE WITH THE CWP IN VERTICAL POSITION
23. KEELHAUL CWP TO UNDER OTEC 10/40 PLATFORM AND PREPARE TO ATTACH PIPE TO PLATFORM
24. RUN HAUL-UP LINE DOWN THROUGH GIMBAL AND HAVE DIVERS ATTACH TO PIPE SUPPORT PENDANT
25. PIPE IS KEELHAULED UNDER PLATFORM
26. DIVER DETACHES LINE
27. DIVERS COMPLETE SECURING OPERATIONS IF REQUIRED
28. HAUL-IN CABLE IS RELEASED
29. BEGIN PUMPING CEMENT THROUGH HOSE INTO STABILIZING TANK
30. ACTIVATE QUICK DISCONNECT ON CEMENT HOSE WHEN STABILIZING CHAMBER IS FULL OF CEMENT
31. RAISE HOSE INTO BARGE
32. TRANSIT TO SHORE ASSEMBLY SITE
33. EMBARK TUGS, BARGES, BOATS FROM SHORE ASSEMBLY SITE TO NEW ORLEANS
34. DEMOBILIZE TUGS, BARGES, AND BOATS AT NEW ORLEANS.

transiting and tow. Figures 4.2-6 and 4.2-7 show preliminary deployment schedules for the two sites. Tables 4.2-2 and 4.2-3 give summaries of the costs for each of these sites.

- ON-SHORE PIPE ASSEMBLY AND BONDING

- PIPE FED INTO LAGOON OR OTHER PROTECTED WATER AREA

- ASSISTED BY PULL FROM TUG
- PROTECTED BY PICKET BOATS
- STABILIZING CABLE FED INTO PIPE AS IT IS ASSEMBLED, IF REQUIRED

Figure 4.2-1. CWP At-Sea Deployment Scenario 1: Rigid CWP

Figure 4.2-2. CWP At-Sea Deployment Scenario 1: Rigid CWP

Figure 4.2-3. CWP At-Sea Deployment Scenario 1: Rigid CWP

• KEELHAULING AND CONNECTING

Figure 4.2-4. CWP At-Sea Deployment Scenario 1: Rigid CWP

Figure 4.2-5. CWP At-Sea Deployment Scenario 1: Rigid CWP

Figure 4.2-6. Scenario 1 Schedule, Brazil

Figure 4.2-7. Scenario 1 Schedule, Puerto Rico

Table 4.2-2. Estimate Summary CWP Deployment for Scenario 1, Brazil

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	1,629.4	604.3	2,233.7
TRANSIT TO SHORE SITE	2,920.6	—	2,920.6
PREDEPLOYMENT AT SHORE SITE	1,370.9	—	1,370.9
TRANSIT TO SITE	364.8	—	364.8
DEPLOYMENT	1,306.9	—	1,306.9
RETURN TRANSIT TO SHORE SITE	364.8	—	364.8
TRANSIT TO NEW ORLEANS	2,920.6	—	2,920.6
DEMOBILIZATION	697.5	—	697.5
TOTAL	11,575.5	604.3	12,179.8

Table 4.2-3. Estimate Summary CWP Deployment for Scenario 1, Puerto Rico

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	1,629.4	604.3	2,233.7
TRANSIT TO SHORE SITE	1,102.6	—	1,102.6
PREDEPLOYMENT AT SHORE SITE	1,370.9	—	1,370.9
TRANSIT TO SITE	60.8	—	60.8
DEPLOYMENT	1,306.9	—	1,306.9
RETURN TRANSIT TO SHORE SITE	60.8	—	60.8
TRANSIT TO NEW ORLEANS	1,102.6	—	1,102.6
DEMOBILIZATION	697.5	—	697.5
TOTAL	7,331.5	604.3	7,935.8

4.2.2.3 Scenario 2: Heavy CWP's

In Scenario 2, the CWP is manufactured on shore into sections that can be loaded onto barges, towed to the assembly site, and connected. The deployment sequence to a vertical position is accomplished at the same time as the sections are joined. Once the CWP is fully deployed and in the vertical position, final connection is made to the transition and platform.

Because of the risk factor associated with construction work at sea, this scenario would be used for CWP's with densities that would not allow the use of Scenario 1. Steel and concrete CWP's will require this deployment scenario because individual, prefabricated sections will weigh up to 300 tons in air. Handling these large, heavy objects at sea will be a difficult, time-consuming process. Mating each new section to those already deployed will require a special construction apparatus that will allow the sections to move together, independent of the platform motions, so that undue stresses will not be introduced into the partially deployed CWP. The assemble apparatus will hold the total deployed CWP until the final connection can be made to the CWP/platform transition. The proposed gimbal and socket transitions of Section 3.4 would be the base for this apparatus in the Brazil platform, and this construction base would remain to become the CWP/platform transition after the assembly apparatus was removed. The Puerto Rico platform would require a separate, converted construction barge to receive the CWP sections for assembly and deployment to a vertical position. The construction barge would also require a construction base and assembly apparatus which would be used only during CWP deployment. Mating procedures of the individual sections would depend on the particular CWP being deployed; the length of time for deployment would depend on the mating method chosen. Welding appears most feasible for steel CWP's, whereas locking joints appear best for the concrete CWP's.

The deployment rate for the concrete CWP is estimated at two sections per 24-hour period. The total assembly time is then 30 days, without any weather contingency factor. The steel CWP requires 24 hours per section.

The special deployment apparatus anticipated for this scenario would be a holding assembly or cradle consisting of columns and slip jacks. The

columns would be supported on a gimbal or socket-type transition which would decouple the CWP motions from the platform. A heavy lift crane would place the prefabricated CWP sections in the cradle and slip jacks would be used for final alignment and mating. After mating, the slip jacks would lower the assembled CWP to the point where another section could be added and the process would repeat.

The heavier CWP's, such as steel and concrete, that will use this deployment method will not require the stabilizing weight necessary for the lighter CWP's. Because of their weight and their deployment from a construction barge at the Puerto Rico site, it may well be necessary to supply the CWP with buoyant collars at that site. These collars would facilitate pipe movement to a position under the platform where final connection would be made to the transition.

Appendix H provides scenarios for Brazil steel deployment and for concrete deployment at both sites. A step-by-step steel CWP deployment for Scenario 2 for Puerto Rico is given in Table 4.2-4; the Brazil scenario would differ as follows:

- The construction barge would not be needed since the OTEC platform itself is the construction deployment vehicle.
- The transit time from the fabrication site to the deployment site would be longer.
- The procedure for keelhauling the deployed CWP from the construction barge to the spar platform would not be needed.

The deployment schedules for concrete differ from steel in that the concrete can be deployed about twice as fast.

Figures 4.2-8 through 4.2-11 show the major sequences for the events during the performance of Scenario 2. Figure 4.2-11 shows the keelhauling of the completely assembled CWP for attachment to the spar platform. Figures 4.2-12 and 4.2-13 show the preliminary deployment schedules for the steel CWP at the two sites; Figures 4.2-14 and 4.2-15 show the schedules for concrete. Tables 4.2-5 through 4.2-8 show the summaries of the costs for these two CWP's at the two sites.

Table 4.2-4. Scenario 2 Procedure

1. MOBILIZE TUGS, BARGES, AND BOATS AT NEW ORLEANS
2. TRANSIT TUGS, BARGES, AND BOATS FROM NEW ORLEANS TO GUAYAMA, PUERTO RICO, SHORE FABRICATION SITE
3. PREPARE TO TRANSPORT FABRICATED CWP SUB-ASSEMBLIES TO SPAR BUOY WHICH WILL HAVE BEEN PREVIOUSLY DEPLOYED AND MOORED
4. AT SHORE SITE, LOAD THIRTY 60-FT CWP SECTIONS ABOARD A 400 X 100-FT BARGE. THIS REQUIRES A 350-TON, 100-FT BOOM TRUCK CRANE OR A 350-TON BRIDGE OR GANTRY CRANE THAT CAN EXTEND TO THE OCEAN SIDE OF THE TRANSPORT BARGE
5. TOW MODIFIED WORK BARGE WITH STIFF-LEG CRANE TO SPAR BUOY. MOOR THE WORK BARGE AT THE DEPLOYMENT SITE
6. MOUNT HARNESS ON CWP SECTION
7. WITH CRANE, LIFT CWP SECTION
8. TRANSFER TO BARGE
9. LOWER TO BARGE
10. RELEASE HARNESS
11. REPEAT FOR 29 MORE SECTIONS
12. LEAVE HARNESS ON LAST CWP SECTION LOADED ONTO BARGE
13. TOW LOADED BARGE TO DEPLOYMENT SITE
14. TIE UP LOADED BARGE TO WORK BARGE
15. WITH WORK BARGE STIFF-LEG CRANE, LIFT CWP SECTION
16. TRANSFER TO WORK BARGE STORAGE AREA
17. LOWER TO WORK BARGE
18. RELEASE HARNESS

Table 4.2-4. Scenario 2 Procedure (Continued)

19. WITH WORK BARGE CRANE, MOVE CWP HARNESS TO TRANSPORT BARGE
20. MOUNT HARNESS ON NEXT CWP SECTION ON TRANSPORT BARGE TO BE TRANSFERRED TO WORK BARGE
21. TRANSPORT BARGE AND TUG MAY HAVE TO STAND BY AS A STORAGE BARGE UNTIL SUFFICIENT AREA IS CLEARED ON THE WORK BARGE TO ACCEPT THE LAST OF THE CWP SECTIONS
22. ASSEMBLE CWP SECTIONS THROUGH THE WORK BARGE MOON-POOL TO MAKE UP 3000-FT LENGTH. (ASSEMBLY SEQUENCE CONTINUES WITH STEP 28.)
23. REPEAT FOR 29 MORE SECTIONS
24. LEAVE HARNESS ON LAST CWP SECTION LOADED ONTO WORK BARGE; A SECOND CWP LIFT HARNESS IS AVAILABLE AT SHORE FABRICATION SITE
25. TOW LIGHT BARGE BACK TO SHORE FABRICATION SITE
26. TRANSPORT SECOND LOAD OF CWP SECTIONS TO SPAR BUOY
27. TRANSPORT BARGE AND TUG MAY HAVE TO STAND BY AS A STORAGE BARGE UNTIL SUFFICIENT AREA IS CLEARED ON THE WORK BARGE TO ACCEPT SECOND LOAD OF CWP SECTIONS
28. WITH WORK BARGE STIFF-LEG CRANE, LIFE CWP SECTION FROM WORK BARGE STORAGE AREA
29. MOVE TO DEPLOYMENT CRADLE
30. POSITION ONTO DEPLOYMENT CRADLE
31. OPEN DEPLOYMENT CRADLE, ALLOWING BOTTOM OF CWP SECTION TO REST ON SLIP-JACK PEDESTALS AT UPPER POSITION
32. LOWER SLIP JACKS 25-FT
33. RELEASE CWP LIFT HARNESS
34. MOVE CRANE TO STORAGE AREA

Table 4.2-4. Scenario 2 Procedure (Continued)

35. MOUNT HARNESS ON NEXT CWP SECTION
36. LIFT CWP SECTION
37. MOVE TO DEPLOYMENT CRADLE
38. POSITION ONTO DEPLOYMENT CRADLE
39. ENGAGE UPPER CWP SECTION WITH HOLDING FIXTURE
40. OPEN DEPLOYMENT CRADLE
41. ADJUST VERTICAL POSITION OF UPPER CWP SECTION SO THAT GAP BETWEEN UPPER AND LOWER SECTIONS IS SET
42. ACTIVATE 12-STATION WELDING SYSTEM
43. RELEASE CWP LIFT HARNESS FROM UPPER CWP SECTION
44. MOVE CRANE TO STORAGE AREA
45. MOUNT HARNESS ON NEXT SECTION
46. RETURN CRANE TO MOONPOOL CENTER
47. MOUNT RING FIXTURE ON TOP OF UPPER CWP SECTION
48. LOWER INNER PLATFORM OF WELDING SYSTEM ON INSIDE OF CWP SECTION; SECURE TO RING FIXTURE
49. BEGIN WELDING CYCLE WITH 12-STATION SYSTEM. SYSTEM ALLOWS FOR 37 PASSES BASED ON NINE REVOLUTIONS AROUND THE CWP. OPERATIONS INCLUDED ARE TACKWELD, WELD AND CHIP, AND CLEAR. IF PRE-HEAT, POST-HEAT, AND/OR STRESS RELIEF ARE REQUIRED, ADDITIONAL STATIONS FOR THESE OPERATIONS CAN BE ADDED. EACH STATION IS ELECTRONICALLY CONTROLLED AND REQUIRES AN OPERATOR AT EACH STATION TO CORRECT FOR MALFUNCTION
50. IN-PROCESS INSPECTION OF WELDS WILL TAKE PLACE AS BEADS ARE COMPLETED
51. PICK UP UNACCEPTABLE WELDS
52. RAISE INNER PLATFORM OF WELDING SYSTEM

Table 4.2-4. Scenario 2 Procedure (Continued)

53. REMOVE RING FIXTURE FROM TOP OF UPPER CWP SECTION;
DISENGAGE GANTRY HOLDING FIXTURE
54. DE-ACTIVATE WELD SYSTEM
55. LOWER SLIP-JACKS 50-FT
56. MOVE STIFF-LEG CRANE BOOM TO STORAGE AREA
57. ENGAGE CWP LIFT HARNESS
58. REPEAT ABOVE ITEMS FOR 58 MORE CWP SECTIONS
59. SECURE AND TIE OFF CWP TO OTEC SPAR BUOY
60. RELOCATE SLIP JACKS SO SUPPORT RING RESTS ON DECK OF
WORK BARGE
61. WITH CRANE, LIFT FLOTATION SECTION INTO PLACE THROUGH
MOONPOOL
62. REMOVE DEPLOYMENT CRADLE
63. REMOVE WELD SYSTEM
64. USING TUGS, GUIDE LINES, TEMPORARY BALLAST, AND DIVERS,
KEELHAUL SPLIT BUOYANT COLLAR SECTIONS INTO PLACE
AROUND CWP BELOW WORK BARGE MOONPOOL
65. SECURE BUOYANT COLLAR AROUND CWP
66. CONTINUE LOWERING CWP USING CABLE PENDANTS FED
THROUGH SLIP-JACK SYSTEM
67. USING DIVERS, SECURE PENDANTS FROM SPAR BUOY AND TUGS
TO FLOTATION
68. CONTINUE LOWERING TOP OF CWP TO A DEPTH OF 200-FT
69. BEGIN KEELHAUL UNDER SPAR BUOY
70. USE CABLES AND WINCHES ON SPAR TO PULL CABLES FED
THROUGH CENTER OF SPAR BUOY

Table 4.2-4. Scenario 2 Procedure (Concluded)

71. WINCHES WILL CONTINUE PULLING AS TUGS PULL AND GUIDE CWP UNDER SPAR BUOY
72. WINCHES WILL PULL THE TRANSITION CONNECTION PIECE AT THE TOP OF THE CWP INTO SPAR BUOY
73. SECURE CWP IN PLACE INSIDE SPAR
74. RELEASE ALL PENDANTS
75. TRANSIT TUGS, BARGES, AND BOATS TO SHORE FABRICATION SITE
76. TRANSIT TUGS, BARGES, AND BOATS TO NEW ORLEANS
77. DEMOBILIZE TUGS, BARGES, AND BOATS AT NEW ORLEANS.

● TOW BARGED CWP SECTIONS TO SITE

Figure 4.2-8. CWP At-Sea Deployment Scenario 2: Heavy CWP

● OFF LOAD TO OTEC OR CONSTRUCTION BARGE

Figure 4.2-9. CWP At-Sea Deployment Scenario 2: Heavy CWP

Figure 4.2-10. CWP Deployment System

- ATTACH FINAL SECTION TO OTEC BARGE
- KEEL HAUL OUT OF CONSTRUCTION MOON POOL AND INTO SPAR

Figure 4.2-11. CWP At-Sea Deployment Scenario 2: Heavy CWP

Figure 4.2-12. Scenario 2 Schedule, Puerto Rico, Steel CWP

Figure 4.2-13. Scenario 2 Schedule, Brazil, Steel CWP

Figure 4.2-14. Scenario 2 Schedule, Puerto Rico, Concrete CWP

Figure 4.2-15. Scenario 2 Schedule, Brazil, Concrete CWP

Table 4.2-5. Estimate Summary Scenario 2, Steel CWP, Puerto Rico

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	203.0	16,062.0	16,265.0
TRANSIT TO SHORE SITE	1,553.1	—	1,553.1
PREDEPLOYMENT AT SHORE SITE	797.0	—	797.0
TRANSIT TO SITE	57.4	—	57.4
DEPLOYMENT	7,313.2	—	7,313.2
RETURN TRANSIT TO SHORE SITE	29.0	—	29.0
TRANSIT TO NEW ORLEANS	1,553.1	—	1,553.1
DEMOBILIZATION	585.2	1,054.0	1,639.2
TOTAL	12,091.0	17,116.0	29,207.0

Table 4.2-6. Estimate Summary Scenario 2, Steel CWP, Brazil

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	114.8	13,737.0	13,851.8
TRANSIT TO SHORE SITE	931.8	—	931.8
PREDEPLOYMENT AT SHORE SITE	598.0	—	598.0
TRANSIT TO SITE	268.8	—	268.8
DEPLOYMENT	5,657.0	—	5,657.0
RETURN TO TRANSIT TO SHORE SITE	98.4	—	98.4
TRANSIT TO NEW ORLEANS	931.8	—	931.8
DEMOBILIZATION	176.4	314.0	490.4
TOTAL	8,777.0	14,051.0	22,828.0

Table 4.2-7. Estimate Summary Scenario 2, Concrete CWP, Puerto Rico

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	220.5	14,624.0	14,844.5
TRANSIT TO SHORE SITE	1,589.7	—	1,589.7
PREDEPLOYMENT AT SHORE SITE	882.0	—	882.0
TRANSIT TO SITE	59.9	—	59.9
DEPLOYMENT	6,443.7	—	6,443.7
RETURN TO TRANSIT TO SHORE SITE	31.5	—	31.5
TRANSIT TO NEW ORLEANS	1,589.7	—	1,589.7
DEMOBILIZATION	620.2	1,054.0	1,674.2
TOTAL	11,437.2	15,678.0	27,115.2

Table 4.2-8. Estimate Summary Scenario 2, Concrete CWP, Brazil

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	114.8	12,299.0	12,413.8
TRANSIT TO SHORE SITE	912.8	—	912.8
PREDEPLOYMENT AT SHORE SITE	731.0	—	731.0
TRANSIT TO SITE	268.8	—	268.8
DEPLOYMENT	4,040.7	—	4,040.7
RETURN TRANSIT TO SHORE SITE	98.4	—	98.4
TRANSIT TO NEW ORLEANS	912.8	—	912.8
DEMOBILIZATION	176.4	314.0	490.4
TOTAL	7,255.7	12,613.0	19,868.7

4.2.2.4 Special Scenario

Because several CWP configurations do not fall into the general categories described for use with Scenarios 1 or 2, the scenarios were adapted to accommodate the stockade, multiple, compliant, and bottom-mounted configurations.

a. Stockade and Multiple Pipe CWP. These two configurations would be designed to be neutrally buoyant and then would use Scenario 1. The design could be accomplished with all the stockade and multiple pipe materials considered. The costs and schedules would be similar to the Scenario 1 deployments discussed in subsection 4.2.3.2, with the exception of the Goldworthy pultrusions for which deployment at sea may prove cost effective. The CWP made up of pultrusions would be loaded into cassetts and onto the deployment vessel. The protrusions would be continuously fed in three layers into a construction gig that would bond the core and the inner and outer layers together. The protrusion core provides the longitudinal strength to the CWP; the inner and outer raps maintain the cross section circular configuration and hoop strength. (More details of this configuration are presented in subsection 4.1.2.) Making the protrusions neutrally buoyant would cost more than the other FRP configurations, but its ability to be continuously assembled in a very rapid time and to deploy in one process may reduce overall costs. Technical problems still exist in the necessary methodology to accomplish this: placing the number of required cassetts on the platform and handling the CWP during assembly and deployment require further study.

b. Compliant CWP. Such compliant CWP's as Bell and Goodyear would use a modified Scenario 2 procedure. The Goodyear compliant CWP sections, which are approximately 100-ft long, would be manufactured in the factory and shipped to the deployment site by barge, collapsed and sitting on end. At the site, the sections would be lifted one at a time with a crane and, still supported by the crane, would be connected to the previously deployed section (Figure 4.2-16). The sections would be deployed through the moon pool of the barge and held by hydraulic gripping jacks at the flange surfaces. Once the new section was attached, the crane would lower the total CWP until the top of the newly attached section could be supported by the jacks. The process would be repeated for each new section.

Figure 4.2-16. CWP Deployment

Once the total CWP was assembled and in the vertical position, final connection would be made to the transition of the OTEC platform as with other Scenario 2 procedures. In this case, however, because of the much lighter CWP, the keelhauling procedure necessary for the Puerto Rico spar attachment would be simplified. The Bell compliant CWP would also use this modified Scenario 2. Both compliant designs require stabilizing weights; the Scenario 1 procedure will be used with the required weight added by pumping cement into the bottom steel container.

The lighter weight and flexible nature of the compliant pipe make the complex deployment cradle of Scenario 2 for the heavier steel and concrete CWP unnecessary. The crane for handling the sections would be sized for all the lifting required, and decoupling the CWP from the platform during deployment would not be required. Preliminary schedules

for the two sites are shown in Figures 4.2-17 and 4.2-18, with cost summaries shown in Tables 4.2-9 and 4.2-10.

Figure 4.2-17. Schedule for Compliant CWP Deployment, Puerto Rico

Figure 4.2-18. Schedule for Compliant CWP Deployment, Brazil

Table 4.2-9. Estimate Summary Deployment, Compliant CWP, Puerto Rico

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	1,003.0	9,920.0	10,923.0
TRANSIT TO SHORE SITE	1,552.6	—	1,552.6
PREDEPLOYMENT AT SHORE SITE	646.3	—	646.3
TRANSIT TO SITE	57.4	—	57.4
DEPLOYMENT	2,474.7	—	2,474.7
RETURN TRANSIT TO SHORE SITE	29.0	—	29.0
TRANSIT TO NEW ORLEANS	1,552.6	—	1,552.6
DEMOBILIZATION	396.2	786.0	1,182.2
TOTAL	7,711.8	10,706.0	18,417.8

Table 4.2-10. Estimate Summary Deployment, Compliant CWP, Brazil

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	914.8	8,595.0	9,509.8
TRANSIT TO SHORE SITE	931.8	—	931.8
PREDEPLOYMENT AT SHORE SITE	476.6	—	476.6
TRANSIT TO SITE	237.0	—	237.0
DEPLOYMENT	1,501.6	—	1,501.6
RETURN TRANSIT TO SHORE SITE	98.4	—	98.4
TRANSIT TO NEW ORLEANS	931.8	—	931.8
DEMOBILIZATION	176.4	46.0	222.4
TOTAL	5,268.4	8,641.0	13,909.4

c. Bottom-Mounted CWP. The bottom-mounted CWP's selected by the screening board were of light material that could be deployed by Scenario 1. This procedure will be used, deviating from Scenario 1 for the supported CWP only in the ballasting and connecting procedure. As discussed in Section 3.4, the bottom-mounted CWP will have a compliant section between the top of the pipe and the platform. This compliant section will be deployed with the CWP, but not connected to the platform until after the ballasting cement is pumped into the anchor chamber. The CWP is then secured to the bottom and in a vertical position. At this time, the platform will be moved over the CWP and the compliant end raised to the platform and connected. It is not anticipated that this procedure's schedule will differ substantially from others of Scenario 1. Figure 4.2-19 gives the preliminary schedule; Table 4.2-11 is the ROM cost summary.

Figure 4.2-19. Schedule for Bottom-Mounted CWP Deployment

Table 4.2-11. Estimate Summary, Deployment, Bottom-Mounted CWP, Puerto Rico

ACTIVITY	COSTS IN \$(K)		
	RECURRING	NON-RECURRING	TOTAL
MOBILIZATION	22,016.8	604.3	22,821.1
TRANSIT TO SHORE SITE	1,089.8	—	1,089.8
PREDEPLOYMENT AT SHORE SITE	1,529.3	—	1,529.3
TRANSIT TO SITE	62.3	—	62.3
DEPLOYMENT	1,553.7	—	1,553.7
RETURN TRANSIT TO SHORE SITE	62.3	—	62.3
TRANSIT TO NEW ORLEANS	1,089.8	—	1,089.8
DEMOBILIZATION	205.0	—	205.0
TOTAL	27,609.0	604.3	28,213.3
200-FT COMPLIANT SECTION	(- 20,700.0)	—	(- 20,700.0)
TOTAL DEPLOYMENT COSTS	6,909.0	604.3	7,513.3

4.2.2.5 Comparison of Scenario Schedules and Costs

A summary of the schedules for the various scenarios, CWP configurations, and locations is shown in Figure 4.2-20, with the ROM costs developed for each. The schedules are divided into three phases: pre-deployment, deployment, and post-deployment. Table 4.2-12 presents the time required to perform the various functions for each of the scenarios.

The pre-deployment time is the time it takes to get the equipment, vessels, and supplies to the actual deployment sites. The mobilization and conversion time required at New Orleans is not included, but the costs associated with these items are. Pre-deployment time consists of item 1 through 3 in Table 4.2-12:

1. Transit from New Orleans to the shore site fabrication site
2. Rig for deployment at the shore site, for tow, assist in fabrication, and on-load as required
3. Transit from shore site to deployment site.

The deployment time is actual time needed at the deployment site, shown as items 4 through 7 of the table.

4. Rigging at the site, including any off-loading required
5. Lowering the CWP either by upending or by assembly of the prefabricated sections
6. Keelhauling and/or final connection to the platform
7. Detachment of deployment equipment and ballasting stabilizing weight, as required.

Post-deployment time is the remainder of the schedule until all equipment and vessels are returned to their home port or disposal site, items 8 through 10:

8. Transit from the deployment site to the shore construction site, and off-loading and on-loading equipment as required
9. Transit back to New Orleans
10. Demobilization time at New Orleans, consisting of returning the vessels to their original configuration, as required.

Figure 4.2-20. Overall Schedules for Deployment Scenarios

The home port for all vessels is assumed to be New Orleans and, while this imposes the penalty of added transit time to the Brazil operations more than to the Puerto Rico site, it is the only apparent pool large enough from which to draw the vessels. These costs can be removed, if, during the preliminary design, the numbers and types of vessels required can be found at the proposed construction sites.

The transit distance and time for the Brazil scenario reflects the distance from Salvador to the operation site for the grazing platform. This longer transit was chosen instead of deploying the CWP just off shore because it would require moving the platform to the operating site with the CWP already connected. It was felt that towing the CWP floating on the surface or barging sections of the CWP at 6 or 7 knots offered a better, less risky solution than moving the OTEC platform the required distance at 1 to 2 knots with the CWP connected.

Table 4.2-12. Schedules/Times and Distances

ITEM	SCENARIO, PUERTO RICO SITE					SCENARIO, BRAZIL SITE			
	LIGHT WEIGHT	STEEL	CONCRETE	COMPLIANT	BOTTOM MOUNT	LIGHT WEIGHT	STEEL	CONCRETE	COMPLIANT
1. TRANSIT (DAYS) N. MI.	19 1700	19 1700	19 1700	19 1700	19 1700	52 4700	52 4700	52 4700	52 4700
2. RIG TOW, ASSIST, AND ON-LOAD	17	10	10	7	18	17	10	10	7
3. TRANSIT TO DEPLOY SITE N. MI.	1 23	1 23	1 23	1 23	1 23	6 510	6 510	6 510	6 510
4. RIG AT SITE	1	2	2	2	1	1	2	2	2
5. LOWERING	6	89	70	22	6	6	89	70	31
6. ATTACHMENT TO PLATFORM	7	7	7	7	7	7	7	7	7
7. BALLAST CLUMP	3	—	—	2	6	3	—	—	2
8. POST DEPLOYMENT TRANSIT N. MI.	1 23	1 23	1 23	1 23	1 23	6 510	6 510	6 510	6 510
9. DEMOBILIZATION TRANSIT N. MI.	19 1700	19 1700	19 1700	19 1700	19 1700	52 4700	52 4700	52 4700	52 4700
10. DEMOBILIZATION AT N.O.	14	14	14	14	14	14	14	14	14
TOTALS (DAYS)	88	162	143	94	92	164	238	219	179

There are two basic differences between the OTEC sites as far as deployment and attachment of the CWP scenarios are concerned:

- The grazing platform required much longer transit time due to its location, and
- The stationary platform required an extra construction barge for deployment of the CWP's using Scenario 2.

As it was, the cost differences varied from \$7.9 million for the deployment of the light-weight Scenario 1 CWP at Puerto Rico, to \$29.2 million for the steel CWP of Scenario 2 at Puerto Rico. The cost difference between like scenarios for the two operating sites varied from \$4.3 to \$7.2 million because of the changes in the scenarios due to the site and platform differences given above.

4.2.3 Conclusions

The scenario used will depend on the site selected for construction, as well as the material and design of the CWP during Phase II. The costs and schedules will also depend on a more extensive survey of vessel availability in ports closer to the deployment site, and to a more detailed look at the vessels involved.

The decision to use one scenario for one type of CWP was made in order to minimize risk, not only to the CWP itself but also risk associated with working and handling equipment at sea. The maximum amount of positive control at all times will produce the minimum risk. Also, the less at-sea assembly required, the less the risk of damage and downtime. The maximum amount of work is therefore performed on-shore or in protected waters, as in Scenario 1. It was also felt that floating the CWP on the surface instead of beneath the surface during Scenario 1 would be less risky, and upending provides more positive control than controlled flooding; with one end of the neutrally buoyant CWP being lowered, the process can be reversed if necessary.

The second best solution, Scenario 2, was chosen for those CWP's where the density of the material would not allow easy upending. An alternate method of upending by using buoyancy cans or chambers either

to be controlled ballasted or with constant buoyancy was not chosen because the cost associated with these options, plus the necessity of removing the buoyancy containers, was considered more risky than assembly of prefabricated sections at the deployment site. Even though this scenario will take more at-sea time, the ability to more positively control the heavy CWP sections was considered a desirable tradeoff. With Scenario 2, the operations can be suspended if the weather makes it difficult to handle the heavy sections.

Since completion of this phase of the work, it was requested that the costs associated with the transit from the construction shore site in Brazil to the operating site be removed. The time to move the grazing plant to its ultimate operating site would not be considered as part of the deployment costs. We have complied with this request and Figure 4.2-21 shows the resulting schedules and costs. Transit time from the construction site to the deployment site was taken as 23 miles for Brazil, the same as for the Puerto Rico scenarios. The overall schedule for the Brazil sites changes only by about 2 weeks, and the alteration in costs is also slight.

Figure 4.2-21. Overall Schedules for Deployment Scenarios With 23-Mile Transit Distance to Construction Site for Brazil Scenarios

4.2.4 Concerns

Several of the at-sea deployments require further investigation during the next phase. These concerns involve the technical aspects of the deployments and operational questions that should be addressed. The technical concerns requiring further investigation are as follows:

- Analysis of stresses developed during the up-ending of a continuous CWP during Scenario 1

- Better definition of handling procedures of CWP sections from transit barge to deployment barge into deployment cradle during Scenario 2 (reach versus load)

- Better definition of the process of keelhauling the heavy CWP under the spar platform

- Better definition of connection procedures for the keelhaunched CWP to both platforms

The operational concerns that need investigating are:

- Cost versus distance to on-shore construction site and the types and number of vessels

- Number and types of vessels required for each scenario

- Better definition of times involved with each scenario.

4.3 REFERENCES

- 4-1 IOC 55152.79-039, "Steel Pipe Coating Costs," M. P. Bianchi to A. B. Griffin, 24 January 1979.
- 4-2 TRW IOC 5515.2.78-446, "Neoprene Coating Summary for OTEC," M. P. Bianchi to L. A. Rosales, September 20, 1978.

5. COST/RISK ANALYSES

The objectives of the cost/risk analyses were to:

- Develop and accumulate the costs and risk assessments associated with the various aspects of the design, fabrication/construction and deployment of each of the candidate CWP design concepts using constant 1978 dollars.

- Display the costs associated with the candidate concepts, as responsive of the DOE Work Breakdown Structure (WBS) and listed for each of the six pipe categories of CWP design; i.e., rigid no joints, rigid articulating joints, compliant, stockade, multiple and bottom mounted.

To satisfy these objectives, each of the CWP candidate concepts described in the preceding sections was evaluated within the following categories:

- Best engineering cost estimates
- Cost uncertainties
- Technical and programmatic risk

Evaluations developed as inputs for concept selection are described in this section.

5.1 BEST ENGINEERING COST ESTIMATES

Estimates of expected costs were developed for each of the candidate concepts, of Figure 5-1, using the following approach:

- Cost estimates were developed from accumulated cost data of our team members, vendor contact, and other related ocean engineering studies as described in the preceding sections.

- Best estimated costs of the candidate concepts were broken down as responsive to the enhanced DOE Work Breakdown Structure described in Appendix D.

- Best estimated costs were also separated and summarized in recurring and non-recurring categories to permit "Potential Benefits" rankings as described in Section 6.

The best estimated costs for candidate concepts are summarily displayed in Figures 5-2 and 5-3 for the Puerto Rico and Brazil sites. The reader is referred to Appendix E for detailed cost breakdown summaries by WBS categories.

A comparison of best estimate costs for the Puerto Rico/spar versus the Brazil barge is presented in Figure 5-4.

5.2 COST UNCERTAINTIES

Cost uncertainties were evaluated in conjunction with the best engineering cost estimates. Ideally, estimates of the CWP system costs should be based on historical cost data from construction of similar structures. In the case of the CWP there is no historical data available. Therefore the best engineering estimates represent a first approximation which must be refined as more design data becomes available. A final accurate cost estimate requires a higher level of information than is presently available or was developed for the concept evaluation phase of the CWP study.

These considerations were incorporated into the cost uncertainty evaluations. Based on historical precedents, it can be assumed that the best engineering cost estimates previously evaluated will involve a number of factors related to uncertainty and/or technical risks as indicated in Figure 5-5. There were three major cost uncertainty elements considered: fabrication/construction, deployment and non-recurring facility costs. Maximum costs were assessed for each of the concepts as indicated in Table 5-1. Detailed evaluations for all concepts are included in Appendix I.

The range of cost uncertainties for all candidates of the Brazil/barge are summarily displayed in Figure 5-6.

5.3 TECHNICAL RISKS

The technical and programmatic risks were assessed by a weighted, category system considering existing technology, extrapolation potential and R&D requirements as indicated in Table 5-2. A sample risk assessment is shown in Table 5-3 and the reader is directed to Appendix J for detailed assessments of all candidates.

The averaged risk evaluations are summarily displayed in Figure 5-7 for all six pipe categories. Critical risk evaluations are displayed in Figure 5-8.

Figure 5-1. Candidate Concepts

Figure 5-2. Best Estimate Costs (Puerto Rico - Spar)

Figure 5-3. Best Estimate Costs (Brazil - Barge)

Figure 5-4. Comparison of Best Estimate Costs for Puerto Rico/Spar Versus Brazil/Barge

Figure 5-5. Cost Estimation: Elements of Cost Uncertainty

Table 5-1. Example of Concept Costing

● CONCEPT 1A: CONCRETE, LIGHTWEIGHT (PUERTO RICO-SPAR)

COST CATEGORY	BEST ESTIMATE COST		COST UNCERTAINTY (MAXIMUM COST EXPECTED)		
	VALUE (\$M)	% TOTAL	RATING	FACTOR	VALUE (\$M)
FAB/CONSTRUCT	8.1	21	LOW	1.15	9.3
DEPLOYMENT	11.4	29	HIGH	1.65	18.8
NON-RECURRING	19.3	50	MEDIUM	1.25	24.1
TOTALS	38.8	100	—	1.35	52.2

Figure 5-6. Cost Uncertainty Brazil - Barge

Table 5-2. Technical Risk Assessment

- RATING SYSTEM PARALLELS DOE/DOC RECOMMENDATION
- CONCEPTS RANKED BY PANEL OF SENIOR TRW/GMDI PERSONNEL ON CWP PROJECT
- EACH CONCEPT RATED IN AREAS OF
 - WALL DESIGN
 - JOINT DESIGN
 - HULL ATTACHMENT
 - FABRICATION/CONSTRUCTION
 - DEPLOYMENT

RATING VALUE	EXISTING TECHNOLOGY	EXTRAPOLATION POTENTIAL		R&D AND TESTING REQ.	HIGH RISK/HIGH DEVELOPMENT COST <
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Table 5-3. Example of Risk Assessment

RATING VALUE	EXISTING TECH	EXTRAPOLATION		R&D AND TESTING
		10/40 MWE	100/400 MWE	
4	LIMITED	NOMINAL	•	HIGH LEVEL
5	WELL-DEVELOPED	•	NOMINAL	•
6	•	LIMITED RISK		NOMINAL

WALL DESIGN RATINGS = 5

- CONCEPT 1B
 - LIGHTWEIGHT CONCRETE
 - SOLID WALL
 - ARTICULATED JOINTS

↑
BEGIN BY SELECTING GENERAL CATEGORY

- • MOVE Laterally TO SELECT BEST RATING
- SHOULD BE GENERAL CORRELATION BETWEEN RATING LEVELS ACROSS CHART

DESIGN FEATURE	RATING	RATIONALE
WALL	5	CONCERNS WITH CREEP, CORROSION STRENGTH, REBAR CONCRETE
JOINTS	3	PAPER DESIGN, EXTRAPOLATION TO 100 MWe SIZE RISKY
HULL ATTACHMENT	4	SHORT DURATION TESTS ONLY; EXTRAPOLATION TO 100 MWe SIZE RISKY
FABRICATION CONSTRUCTION	8	SOME R&D FOR AT-SEA OPERATION
DEPLOYMENT	2	LIMITED TECHNOLOGY, HIGH-RISK EXTRAPOLATION, AND SIGNIFICANT R&D REQUIREMENTS
TOTAL	22	

• AVERAGE = 4.8; RANGE = 3 - 8

- AVERAGE RISK RATINGS RANGE FROM 3 TO 8
- UNFEASIBLE (RATINGS 1-2) CONCEPTS ELIMINATED DURING CONCEPT SCREENING
- ALL CONCEPTS INVOLVE SOME AMOUNT OF EXTRAPOLATION AND RISK (NO 9-10 RATINGS)
- RIGID (SANDWICH WALL) FRP HAS LOWEST RISK
- ARTICULATED JOINT DESIGNS (CONCRETE AND STEEL) AND BOTTOM MOUNTED CONCEPTS ARE HIGHEST RISK CONCEPTS
- POLYETHYLENE CONCEPTS ARE HIGHER RISK THAN FRP IN ALL COMMON CWP CATEGORIES

Figure 5-7. Concept Risk Evaluation Averaged Risks

Figure 5-8. Critical Risk Rating Critical Risk

6. CONCEPT SELECTION FOR PRELIMINARY DESIGN

Candidate CWP concept ranking was based on the cost estimates and technical risk evaluations described in the preceding section. A straightforward scheme was used to combine these two factors into a "Figure-of-Demerit" for use in the final rankings. Sensitivity of ranking results to assumptions (e.g., relative weighting of risk versus cost) and certain qualitative concept design evaluations have also been used in making the final selections.

6.1 METHODOLOGY

The key issue of concept ranking is to determine a quantitative measure of relative concept merit for distinguishing the "best" concepts. The two fundamental aspects of concept merit, system cost and technical risk, are independent factors that must be combined to make direct comparisons of concepts.

A map of cost versus risk for three hypothetical CWP concepts is presented in Figure 6-1. As presented in the figure, concept cost and risk are plotted on increasing scales along orthogonal axes; the relative distance from the origin represents some combined measure of the concept desirability, and the shortest distance is "best" in terms of lowest cost and risk. The map also illustrates the fundamental question to be addressed in developing the ranking scheme: what is the relative importance (i.e., weighting) of risk versus cost to be used. The effect of relative factor weighting is to adjust the scaling of the axes so that the corresponding factors have increased or decreased impact on the total distance measure. This total distance measure is then associated with a concept Figure-of-Demerit (FOD) which is defined below in more specific mathematical terms.

The concept FOD used in the TRW ranking analysis is calculated based on a given concept cost, C , and risk factor, R , as developed in Section 5. First, a normalized cost relative to the maximum cost of a given concept ($C_{\max}$) is calculated

$$\hat{C} = \frac{C}{C_{\max}}$$

COMPOSITE EFFECT
RELATED TO RELATIVE
DISTANCE TO ORIGIN

- SHORTEST DISTANCE
IS LOWEST RISK /
LOWEST COST

NEED TO SCALE THE COST/
RISK AXIS TO REFLECT
RELATIVE IMPORTANCE, e.g.,

- EXPANDING RISK AXIS
SCALE EQUIVALENT TO
INCREASING RELATIVE
IMPORTANCE TO TOTAL
DISTANCE MEASURE

DISTANCE = COMPOSITE
FIGURE-OF-DEMERIT (FOD)

- FUNCTION OF RELATIVE
COST/RISK SCALING, AS
WELL AS ACTUAL VALUES
OF COST AND RISK FACTORS

Figure 6-1. Concept Mapping: Cost Versus Risk Factors

Similarly, a normalized risk is defined, but the risk scale used in Section 5 is first modified so that the higher risk concepts correspond to higher risk factor values, which is opposite to that in the preceding section. The equations for calculating the modified risk factor, R' , and its corresponding normalized value are:

$$R' = 10 - R$$

$$\hat{R} = \frac{R'}{R'_{\max}}$$

Note that both $\hat{C}$ and $\hat{R}$ can vary from 0 to 1.0 and provide relative measures of cost and risk, respectively, for any one concept.

Concept FOD is now defined as a linear combination of the normalized cost and risk factors using a risk weighting or scaling factor, S_R , which can vary from 0 to 1. Relative cost scaling is then $(1 - S_R)$ and FOD is given by the following simple linear relation:

$$FOD = S_R \cdot \hat{R} + (1 - S_R) \hat{C}$$

This relation combines the cost/risk factors as a function of the assumed scaling or weighting factor S_R . S_R varying from 0 to 1 corresponds to going from cost-only weighting to risk-only weighting.

An FOD plot for the three hypothetical concepts shown in Figure 6-1 is presented in Figure 6-2. The figure illustrates the functional relationship of FOD to S_R and the consequences to concept rankings. The minimum FOD line among the three concepts plotted defines the best concept. At any crossing of the FOD lines, the best concept will change so that different rankings result over different ranges of S_R .

Figure 6-2. Use of Figure-of-Demerit for Concept Ranking

The TRW approach to FOD calculations was to select a nominal cost/risk scaling and to then determine the sensitivity of the corresponding baseline ranking of the CWP concepts. A baseline ranking was determined by integrating the FOD versus S_R plots for each concept and comparing the integrated or average FOD values. The best estimates of system costs were used in evaluation $\hat{C}$ and the composite risk factors used for determining $\hat{R}$.

The baseline ranking results, as well as other ranking scenarios and sensitivity analyses used in determining final concept ranking, are described below.

6.2 CONCEPT RANKING RESULTS

Baseline CWP concept rankings are presented in Figure 6-3 and show the FOD calculated for each. As indicated in subsection 6.1, these results correspond to best estimate costs (the spar hull was assumed) and composite risk ratings. FRP designs represent the four top-ranked concepts. The compliant Goodyear elastomer-type design is ranked third. The top two designs are the sandwich and solid wall rigid configurations; stockade and multiple pipe FRP configurations are ranked third and fourth, respectively.

Grouped below the top four is a set of five distinct designs: the Bell elastomer concept, the multiple polyethylene concept, the bottom-mounted FRP concept, the Bell pressurized compliant wall concept, and the polyethylene stockade concept. Rigid concrete and steel designs, along with bottom-mounted designs of elastomer and polyethylene, make up the last group in the ranking list.

The baseline ranking list shown in Figure 6-3 was considered further in light of any special features of the concepts that may have been "lost" in the baseline ranking process. Separate cost-only and risk-only rankings were developed for this purpose to see if significant ranking differences result. While it is felt that concept selection should be based on consideration of both factors, the implications of the separate cost or risk factors may be of help in adjusting the baseline ranking for concepts with tightly grouped baseline FOD values.

Separate cost-only rankings were developed corresponding to best estimate costs, recurring costs (i.e., neglect non-recurring items which would be reduced in production item costing according to the number produced), and maximum estimated costs. Risk-only rankings were also developed for the composite and critical item risk ratings. A brief summary of major differences found when compared with baseline rankings are as follows:

- Recurring Cost-Only Rankings. Concrete CWP designs would be ranked significantly higher (number 4 for the lightweight and number 5 for mediumweight, wall-supported designs)

- Maximum Cost-Only Rankings. Bell elastomer-type designs would be dropped down to number 13 from number 8 in the baseline rankings

- Composite Risk-Only Rankings. Rigid steel would be rated 4 instead of 15

- Critical Item Risk Only Rankings. All bottom-mounted concepts would be rated last because of the problem of attachment to the hull.

Figure 6-3. Baseline Rankings

Noting the potential importance of these effects in making final ranking adjustments, two special scenarios were postulated to further consider the sensitivity of baseline ranking results:

1. Conservative Scenario. Consider worst-case risk as the relevant factor corresponding to a weakest link-type argument for CWP system development failure. Additionally, use worst-case cost, but weight the risk factor more strongly than cost ($S_R = 0.75$ was assumed).

2. Potential Benefits Scenario. Consider best estimate recurring cost as the overriding factor in the evaluations corresponding to the eventual production of many CWP's for OTEC and effective amortization of non-recurring cost items. Weight composite risk as a relatively minor factor ($S_R = 0.25$ was assumed).

The resulting rankings for these two scenarios are given in Table 6-1. Comparison with the baseline rankings provides additional support for scoring the concrete concepts higher. The concrete concepts are relatively conservative development risks compared to the bottom-mounted and polyethylene stockade designs, and they have potential low-cost benefits for other than a one item production basis.

Based on the above results, a final CWP concept ranking was generated and is shown in Table 6-2. The only significant adjustment from the baseline ranking was to move up the medium-weight concrete design (with articulated joints) from 10 to 7. The final rankings do not include the steel designs or the bottom-mounted elastomer and polyethylene designs which were grouped last in the baseline results and had no identified advantages over the designs included.

Table 6-1. Effect of Conservative and Potential Benefit Scenarios on Concept Rankings

BASILINE RANKING	CONCEPT	CONSERVATIVE SCENARIO RANK	POTENTIAL BENEFITS SCENARIO RANK
1	FRP, SANDWICH	1	1
2	FRP, SOLID	3	2
3	STOCKADE, FRP	2	3
4	MULTIPLE, FRP	4	4
5	ELASTOMER (GOODYEAR)	5	7
6	MULTIPLE, POLYETHYLENE	6	9
7	BOTTOM-MOUNTED, FRP	13	12
8	ELASTOMER (BELL)	7	11
9	STOCKADE, POLYETHYLENE	11	10
10	CONCRETE, ARTICULATED JOINTS, MEDIUM WEIGHT	9	6
11	CONCRETE, ARTICULATED JOINTS, LIGHT WEIGHT	8	5
12	CONCRETE, ARTICULATED JOINTS, LIGHT WEIGHT, CENTER SUPPORT	10	8
13	STEEL, ARTICULATED JOINTS	12	13
14	BOTTOM-MOUNTED, ELASTOMER (GOODYEAR)	14	14
15	STEEL, RIGID JOINTS	16	16
16	BOTTOM-MOUNTED POLYETHYLENE	15	15

Table 6-2. Final Rankings

FINAL RANKING	CONCEPT DESCRIPTION		INTEGRATED COST/RISK		COMMENTS
	CATEGORY	DESIGN	RANK	FOD	
1	RIGID	FRP, SANDWICH	1	1.00	• BEST COST/RISK RANKING
2	RIGID	FRP, SOLID WALL	2	1.15	
3	STOCKADE	FRP	3	1.32	
4	MULTIPLE	FRP	4	1.38	
5	COMPLIANT	CABLE REINFORCED ELASTOMER	5	1.84	
6	MULTIPLE	POLYETHYLENE	6	1.72	
7	RIGID-ARTICULATED JOINTS	CONCRETE MEDIUM WEIGHT	10	2.05	• LOW RECURRING COSTS • LIGHTWEIGHT, CENTER SUPPORT AS DESIGN OPTIONS
8	BOTTOM MOUNTED	FRP	7	1.75	• HIGH CRITICAL RISK (HULL ATTACHMENT) LIMITATIONS
9	COMPLIANT	DOUBLE WALL PRESSURIZED REINFORCED ELASTOMER	8	1.98	
10	STOCKADE	POLYETHYLENE	9	2.02	