A Conceptual Design for a Semi-submerged Offshore Nuclear Generating Station

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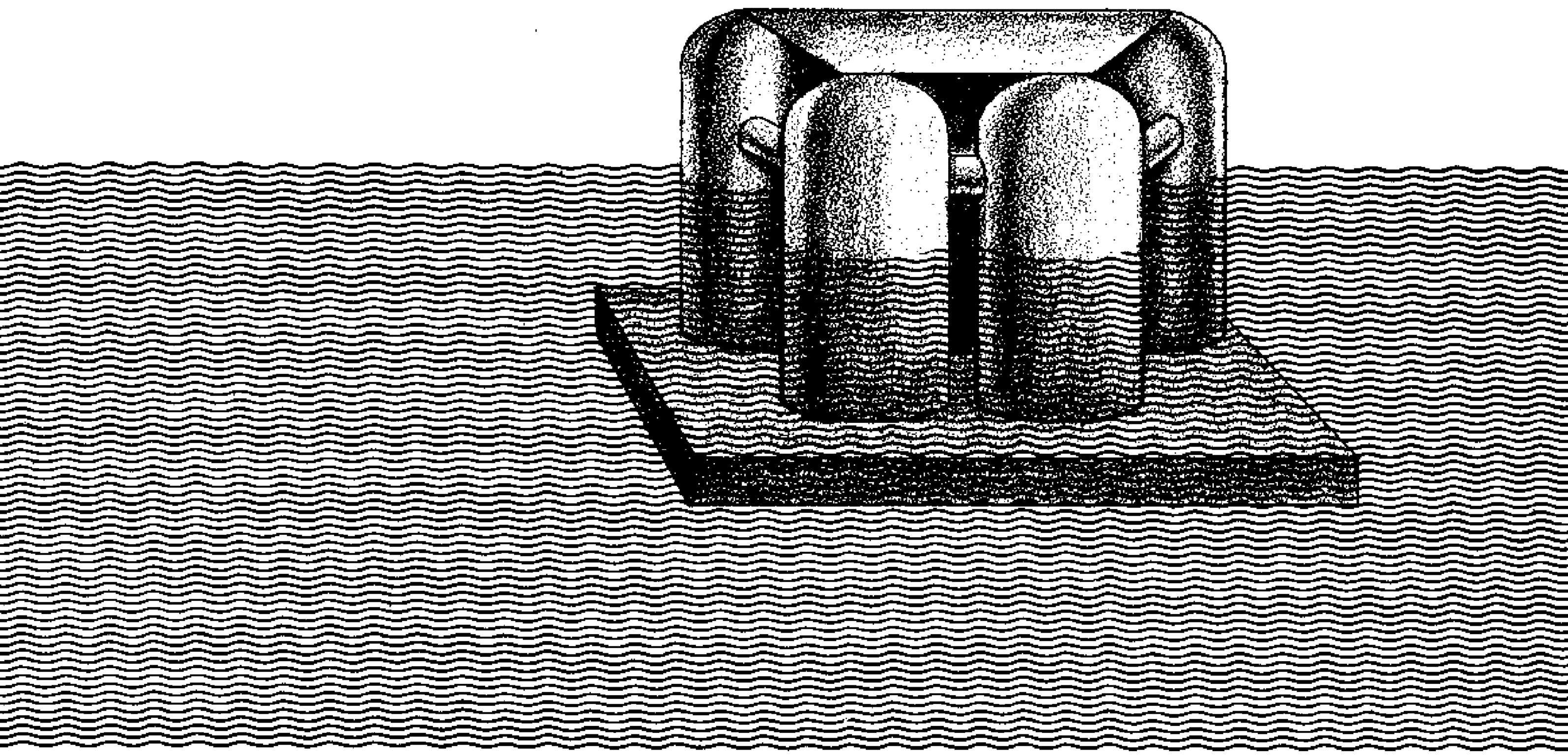

Ocean Engineering Design Project
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CE 573—Ocean Engineering Design
Oregon State University
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A CONCEPTUAL DESIGN FOR A SEMI-SUBMERGED OFFSHORE NUCLEAR GENERATING STATION

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PREFACE

Ocean engineering involves the application of the tools of traditional engineering disciplines for planning, design, construction, inspection and maintenance of systems used for developing or protecting the oceanic environment. These systems may be employed in the estuarine, nearshore or coastal environments as well as on the surface, intermediate waters, bottom and subbottom of the ocean.

To promote greater understanding of oceanic engineering and systems engineering concepts at Oregon State University, futuristic and realistic design problems are assigned to the Ocean Engineering Design class, CE 573. During the 1971 spring term, the design problem concerned alternative sitings of a near shore nuclear power plant.

Students enrolled in the 1971 Ocean Engineering Design class had backgrounds and experience relating to electrical engineering, nuclear submarine duty, mechanical engineering, oceanography, aviation, ocean engineering, environmental engineering, structural engineering and coastal engineering. The design approach was on a group basis following basic lectures and completion of assigned coastal engineering problem sets. The students' interests and talents were exploited in proceeding with the design problem. They were also exposed to unfamiliar engineering material that required careful study prior to application in a design situation.

This approach is consistent with the philosophy that an OSU ocean engineer must first be a specialist in a particular engineering discipline (chemical, civil, electrical or mechanical engineering). He is then encouraged to become a generalist in order to practice his profession in the ocean environment.

The contents of this report do not necessarily reflect the views and policies of the School of Engineering, Oregon State University The mention of trade names or commercial products does not imply endorsement or recommendation for their use.

The mention of nuclear power development in Oregon often evokes emotional outbursts from various interest groups. This case problem was entirely hypothetical; this is to disclaim any implied intent to favor and/or discount sitings of nuclear power plants over other power resources. This work was not intended to be all inclusive, for the time available for this project was of the order of six weeks for only three term-credits. The students involved in this design project presented their findings in seminar to faculty, consulting engineers and professionals interested in the developing field of ocean engineering. The cooperative and enthusiastic support of these participants of this program is sincerely appreciated.

It is hoped the reader will find the following treatment of design considerations for the near shore siting of nuclear power plants of interest.

Larry S. Slotta
Associate Professor

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1. Introduction

How much will the electrical power requirements of the Pacific Northwest increase during the remainder of this century and how shall we meet this new demand with a minimum alteration of the environment? Since our present technological capabilities provide us with a limited number of acceptable alternatives, this is undoubtedly one of the most challenging questions facing our society today.

1.1 WHY NUCLEAR POWER?

The 1968 peak electrical load in the Pacific Northwest was 16,500 megawatts (MWe), with a 1990 load of 55,600 MWe predicted. This is an annual growth rate of 5.5 percent/year, which is slightly below the national average of 7 percent/year.[1]* In their January 1969 report, the Bonneville Power Administration concluded that increasing our hydroelectric capability to the practical limit would not be sufficient to meet the increase in firm energy demand.[2] They also concluded that fossil -fueled thermal plants, with a few exceptions, are not economical in this area due to a paucity of naturally occurring fuels. The result of these conclusions is that it will be necessary to construct nuclear power plants to supply about 20,000 MWe of the increased base load of 1990, with new and existing hydro-electric projects supplying peaking loads.

1.2 THE SITING PROBLEM

Where will these 18 to 20 nuclear power plants be located? This is another complex question, for the answer depends on a wide spectrum of factors, such as: distribution of load; availability of land; geological soundness and seismic stability of the site; availability of cooling water; local hydrological and meteorological conditions; environmental effects of waste heat rejection and release of radioactive materials; genetic and somatic effects of long-term exposure to low levels of ionizing radiation; technological feasibility of plant construction and operation; economic practicability; aesthetic desirability; etc. To date, government agencies and utilities have often been unable to present acceptable solutions to one or more of these aspects of the site selection problem. This, in turn, has lead to several plant delays and/or cancellations. If the predicted demands of the future are to be met on time, sites for these plants that are acceptable to both the utilities <u>and</u> the public must be found without delay.

1.3 A POSSIBLE SOLUTION

In this report, the feasibility and practicability of siting a nuclear power plant off the Oregon coast is examined. The basic design

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*Superscript indicates references found at the end of the report.

considered is a 1,100 MWe High Temperature Gas Cooled Reactor plant that is to be floated into position aboard a large, prestressed concrete barge and partially submerged in 150 feet of water. The site is located approximately 2 1/2 miles due West of Big Creek, Oregon on Heceta Bank. Single-pass cooling is utilized for heat rejection with the electrical power being transmitted ashore through a submerged cable. Personnel and equipment access will be by helicopter, small boat and barge, with limited harbor facilities at Big Creek. Figure 1-1 illustrates the location of the proposed site.

1.4 SCOPE OF THE REPORT

This study was undertaken as partial fulfillment of the requirements for a three credit hour graduate course at Oregon State University (CE573, Ocean Engineering Design). The limited scope of the project did not include a detailed design study. Rather, our objectives were:

1. To determine if the concept of deep water offshore siting of nuclear power plants was feasible, practical and desirable.

2. To make reasonable estimates of the major design parameters.

3. To suggest areas in which further research would be required.

It should be emphasized that the suggested location and design characteristics of the plant represent only one possible solution to the problem. There are undoubtedly many offshore locations that might prove to be more desirable than the one selected. There are also countless other configurations for the hull, the barge, the plant layout, the foundation, etc., which might be closer to the optimum than the ones presented in this report. This conceptual design is offered only as an attempt to illustrate the order of magnitude of the engineering challenge involved in offshore siting of large nuclear power plants.

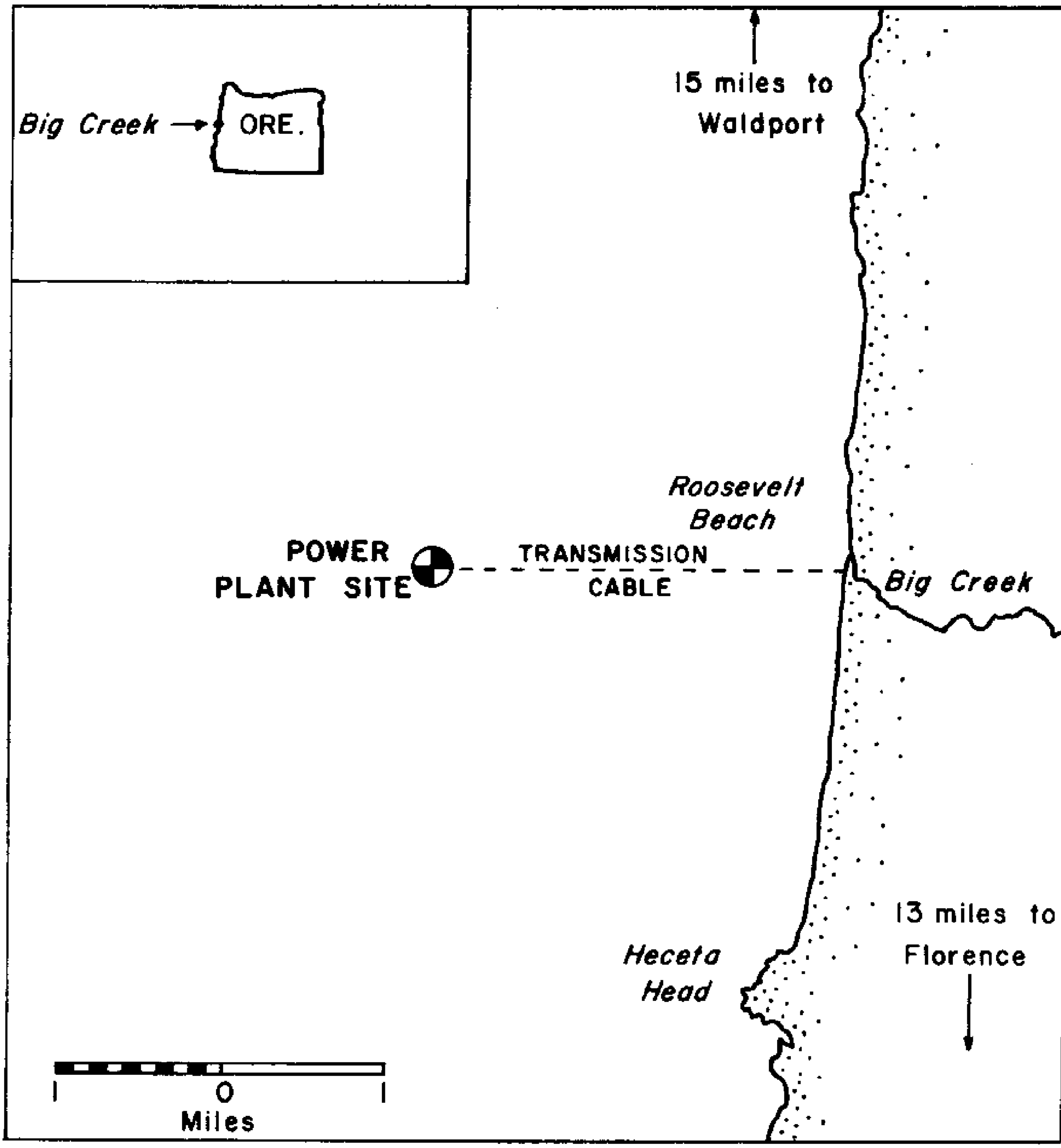


Figure 1-1. Location of proposed site for an offshore nuclear power plant.

2. Offshore Site Selection Criteria

Selection of a site for a nuclear power plant is a complex process involving a wide range of considerations and constraints. As with any selection process, the optimum choice is the one which maximizes the number of desirable features, minimizes the number of undesirable features, and still complies with all externally imposed restrictions. After reviewing the various features of large nuclear power plants and the multitude of constraints associated with them, we concluded that an offshore site, if feasible, might prove to be optimum.

The criteria considered in making the decision to further explore the feasibility of offshore siting have been grouped into four categories: Effects of the plant on the environment; Site suitability and availability; Social and legal restrictions; Economics of construction and operation.

2.1 EFFECTS OF THE PLANT ON THE ENVIRONMENT

Public awareness of and concern for environmental quality, priority of land uses, and public safety is making it increasingly difficult to find acceptable, let alone optimum, locations for nuclear power plants.[3] Hence, one of the most important factors that must be considered in site selection is how the plant will affect the surrounding environment and nearby populace. The most serious environmental problems associated with large nuclear plants are thermal effects, radiation effects, and nuclear accident potential. Each of these categories was examined in comparing the desirability of offshore nuclear plant siting with conventional siting on dry land.

2.1.1 Thermal Effects

All large thermal power plants reject considerable quantities of waste heat to the environment. A dry land site must initially reject this heat to water from a river, a lake, an estuary, or the ocean, depending on the location of the plant. Since the discharge of high temperature effluents directly into rivers, lakes and estuaries is undesirable (and in most states prohibited) because of adverse effects on the biota, either huge cooling towers or cooling ponds are usually employed. Coastal plants must pump large volumes of water from the ocean and return the hot water in a manner which precludes significant biological damage.

The location of a plant offshore in relatively deep water, on the other hand, provides abundant quantities of low temperature cooling water along with the possibility of discharging the heated effluent directly to the environment with a minimum of detrimental side effects. (See Chapter 4)

2.1.2 Radiation Effects

In recent years, a heated controversy has developed concerning the potential somatic and genetic effects of long term exposure to very small increases in background ionizing radiation from nuclear power plants. Drs. John W. Gofman and Arthur R. Tamplin have alleged that exposure to an additional 170 millirem (mr) per year (Federal Radiation Council guideline limit for nuclear power plant source) would result in 32,000 additional cancer and leukemia deaths per year.[4] The nuclear power industry and the U. S. Atomic Energy Commission counter with the assertion that the average population dose from existing nuclear plants (.01 mr/yr) is a fraction of the increase in ionizing radiation dosage from cosmic rays that one would encounter moving from sea level to Denver, Colorado.[5]

Regardless of the relative merits and validity of these opposing claims, two things are certain:

1. The public will continue to be very skeptical about nuclear power plants until more is known about the effects of these small increases in background radiation, and

2. Locating nuclear power plants offshore (and particularly in a semi-submerged condition) reduces their contribution to background radiation in populated areas to essentially zero.

The choice of a High Temperature Gas Cooled Reactor eliminates, for all practical purposes, the discharge of radioactive effluents into the environment.[6]

2.1.3 Nuclear Accident Potential

As a result of very rigid licensing and operating restrictions imposed by the AEC and other regulatory agencies, the probability of a serious nuclear accident occurring is extremely small.[7] However, in spite of this low probability of occurrance, large numbers of nuclear power plants located in populated areas represent a finite and unacceptable potential for disaster. This has resulted in dry land sites that are generally located in fairly remote areas, far from the load centers they serve.

The offshore plant, while subject to about the same probability of accident occurrence, represents less of a danger potential to the public. First, because of the availability of a nearly continuous strip of suitable offshore real estate, the plant can be located at a point which is a safe distance from major population centers and at the same time is reasonably close to the load center. Secondly, since an earthquake is the most severe natural disaster (from a nuclear accident standpoint), the ability to fully or at lease partially de-couple the plant from the sea floor significantly reduces the danger potential. Finally, the containment of the entire plant in a prestressed concrete hull partially surrounded by water further protects the public in the event a serious accident were to occur.

2.2 SITE SUITABILITY AND AVAILABILITY

The next problem to be solved, once you have found a region where the environmental impacts of the plant are acceptable, is to find an actual site that is both suitable for location of a plant and is available.

2.2.1 Site Suitability

In order for a location to be considered "suitable" for siting a nuclear power plant, the nature of the surrounding physical environment and its potential effects on the plant must be known in considerable detail. This is primarily a reactor safety requirement, since it must be determined conclusively that it is feasible to construct a plant that will withstand the most severe conditions that could conceivably occur. Included in this requirement would be knowledge of meteorological conditions (inversions, prevailing wind data, frequency and severity of hurricanes or tornadoes, etc.), geological conditions (soil mechanics, location and nature of bedrock, etc.), geophysical conditions (seismic history, current tectonic activity, maximum probable earthquake intensity, etc.) and hydrological/oceanographic conditions (availability of cooling water, river flow rates, tidal currents, tsunamis, etc.).[8]

It is difficult to assess the relative suitability of dry land and offshore sites in detail, since the characteristics of their respective environments are quite different. In some respects the dry land site would have obvious advantages, such as accessibility of the site for construction and operation, and a surrounding environment that is generally less harsh. On the other hand, a plant located in fairly deep water (say greater than 100') would be subjected to a smaller range of environmental perturbations than would a plant located on the shoreline.

2.2.2 Site Availability

There is little doubt that offshore siting would be preferable to dry land siting from a site availability standpoint. The cost of a piece of land that is both big enough to accommodate a nuclear power plant and is suitable as a site varies considerably, depending on the region. However, the trend toward increased land prices is consistant throughout the country, and is particularly noticeable in areas of higher population density.

In addition to cost factors, competition with other real estate interests would essentially be eliminated by moving offshore, thereby significantly reducing one of the major obstacles to public acceptance of nuclear power plants.

2.3 SOCIAL AND LEGAL RESTRICTIONS

As public awareness of widespread environmental degradation mounted during the last decade, so did the number of laws, regulations, and standards concerned with improving the quality of the environment.

2.3.1 Thermal Pollution Control

The area of thermal pollution from central station thermal power plants was not overlooked in this recent proliferation of environmental legislation. Since 1968, all fifty states have proposed maximum temperature and temperature rise criteria for interstate waters.[9]

Temperature standards for Oregon are outlined in Table 2-I (p.9).

In spite of these seemingly specific standards, there is a great deal of uncertainty involved in their application. In general, the criteria for a given body of water depend on the classification of that body and the exact location where the requirements must be met. The numbers presented in Table 2-I are the maximum allowable temperatures in the water body and the maximum temperature rise that can result from the addition of heated effluents. However, it should be noted that these measurements are generally to be made at a point some unspecified distance away from the point of discharge (i.e. outside a 'mixing zone'). In theory, this distance can range from zero to infinity. The regulations concerning mixing zones in Oregon are complex, with dimensions of the zones usually being determined on a case-by-case agreement between the Federal Water Pollution Control Administration and the state Department of Environmental Quality.[10]

Maximum * Temperature (°F)	Maximum Temperature Rise (°F)	Water Body
64	2	Generally
70	2	Multnomah Channel, Willamette and Snake Rivers
68	2	Columbia, Grande Ronde and Walla Walla Rivers
72	2	Klamath River
	0	Marine Water
*Any measurable increase in temperature when the receiving water temperatures are 64°F or above, or more than 2°F increase when receiving water temperatures are 62°F or less are prohibited.		

Table 2-I Water temperature standards for Oregon.

These water quality standards essentially rule out the use of Oregon's inland waters for single-pass cooling of large (1,000 MWe) thermal power plants. They do not, however, preclude single pass cooling with salt water at open ocean sites.[11]

2.3.2 Ionizing Radiation Controls

There are two principal federal agencies charged with the responsibility of protecting the general public from harmful radiation side effects; the Federal Radiation Council (FRC) and Atomic Energy Commission (AEC). The FRC has the task of setting standards for "permissible" levels of exposure of the public to ionizing radiation. This includes recommending limits on whole body exposure to radiation, recommending limits on concentrations of various nuclides in natural waters, etc. The FRC's guidelines are in turn used by the AEC to establish or modify their rules and regulations pertaining to radiation from nuclear power plants. These rules are implemented in the following three ways:

> 1. The AEC establishes and publishes in the Federal Register rules and procedures for locating, designing and operating a nuclear power plant.
>
> 2. The AEC conducts detailed safety review prior to the issuance of a construction permit and again, prior to the issuance of an operating permit...to ensure that the facility is located, designed, engineered and constructed in strict conformance with AEC regulations and with well-disciplined quality assurance and engineering practices.
>
> 3. Thirdly, the AEC inspects the licensed project after it is in operation to assure compliance with AEC regulations.[12]

To be a little more specific about the actual limits, they "are established by conservatively computing the release rate which, at the point of highest radiation level averaged over a period of one year, on or beyond the site boundary, would result in an exposure to the whole body of a hypothetical individual, present throughout the year at the point of highest radiation level, equal to the radiation protection guide for an individual in the population recommended by the Federal Radiation Council (500 mrem/year)."[13] Clearly, a plant located over two miles offshore, semi-submerged in deep water will have no difficulty meeting limits based on this standard.

2.3.3 Offshore Legal Restrictions

The legal aspects of siting a nuclear power plant offshore are not clearly spelled out, although there doesn't appear to be any obvious obstacles. Locating the plant within the territorial seas of the United States (within 3 miles of shore) would be mandatory. Under the Submerged Lands Act of 1953, the land under the territorial seas is deeded to the state whose coastline borders the sea.[14] Thus, negotiations for the right to use a piece of land for a nuclear power plant site would be between the company involved and the State of Oregon. Since conflicting uses of land that is 100' to 150' underwater are not commonplace, no real legal difficulties are envisioned.

2.3.4 Social Restrictions

The social impact of increasing the electrical power generating capabilities of the Pacific Northwest is not really a factor in deciding between dry land and offshore sites. Construction of either type of plant will enhance economic activity and attract people to the area. An enlarged population and economic growth would then combine to add additional sources of pollution and other undesireable side effects to those directly attributable to the generation of the electricity. If, however, a plant must be built, locating it offshore has several social advantages. The factors already discussed concerning increased public safety and decreased land use conflicts would make the offshore plant more socially acceptable. In addition, an offshore, semi-submerged plant would tend to be less aestheticly obtrusive than its conventionally monstrous land based counterpart.

2.4 ECONOMICS OF CONSTRUCTION AND OPERATION

A quick look at the relative cost of constructing and operating dry land and offshore nuclear plants indicates that an offshore plant could be competitive.

2.4.1 Construction Costs

A detailed breakdown of all costs related to the construction of large land based nuclear power plants is not readily available, but an approximate figure is $245.00 per kilowatt of generating capability[15] (i.e. an 1100 MWe plant would cost about 270 million 1970 dollars). This includes significant expenditures for site selection studies, for extensive safeguard features, and for large cooling towers/ponds. In addition, the plant designs and site layouts vary considerably from one plant to another, requiring a large design expenditure on each plant.

The offshore plant, by comparison, would not require expenditure of vast sums for land or for an elaborate cooling system, and could involve somewhat lower safeguard costs. Also, many plants of the same basic design could be constructed in a large graving dock and towed to different sites when completed. The advantage of year-round construction, assembly line techniques, efficient labor utilization, etc. would further reduce design and construction costs per plant.[3] Site preparation, however, would probably be more costly in the offshore case.

2.4.2 Operating Costs

The operating costs of the two types of plant should be comparable. A study made by Bowers and Wichner[16] compared the cost of supplying power to an inland city from an offshore plant and supplying it from a near-load facility. The results of their study indicated that the lower cost of energy at the bus bar of the offshore plant would be offset by the increased transmission costs (up to 80 miles, including 10 miles of submerged cable). Since their study considered a light water reactor, the results would be conservative when considering the more efficient HTGR.

Another factor to be considered in comparing operating costs is salaries of personnel. It is doubtful that a conventional scheme for rotation of plant operations personnel would be suitable for an offshore site, in light of the relatively complicated transportation arrangements involved. A plan which rotated pairs of crews on a 4-day-on/4-day-off basis would probably be more effective. This would, in turn, result in a larger total number of operators and a proportionate increase in operating costs.

3. Environmental Parameters

In a conceptual design such as this it is necessary to assess fully the parameters of the environment which might affect or be affected by the design under study. These effects are integral parts of the design and site choice considerations.

This study, involving the conceptual design and siting of a nuclear power generating plant in the nearshore regime off the coast of Oregon included a study of the nearshore environmental parameters such as waves, winds, currents, geology/geophysics, and biological/chemical oceanography.

Since this is a conceptual study, the intent was to assess the environmental parameters in a general sense, attempting to show conditions which might be typical of many nearshore regions of the Oregon Coast. No attempt was made to carry out a rigorous study of a specific site.

3.1 WAVES AND SWELL

Few actual deepwater wave observations exist in the Pacific Northwest. This necessitates the use of hindcast methods to predict the wave regime to be expected in the area of interest. A detailed study of the waves of the Pacific Northwest was conducted by National Marine Consultants in 1960-61.[17] The meteorological data from 1956-58 was considered collectively to obtain data representing an "average" year. The meteorological data was analyzed to obtain the statistical heights and periods to be expected as a result of the application of the hindcasting techniques employing the spectral energy method of Pierson, Neumann, and James.[18] The results are shown in Table 3-I for three stations off the Oregon coast so as to include the complete area of interest in this study as shown in Figure 3-1.

On the north Pacific Coast of the United States the weather pattern is relatively uniform and storms of intense severity are likely to occur several times each year. With this knowledge one may utilize the statistical significant wave height obtained by the hindcasting technique to predict a design wave. The significant wave heights in Table 3-I are in themselves averages, so one can assume that there exists within the spectrum a maximum wave of much greater height. It is usual to design a structure for the maximum possible wave for the accepted frequency of storm and therefore a relationship must be found between H_{sig} and H_{max} in order to establish the design wave height.[19]

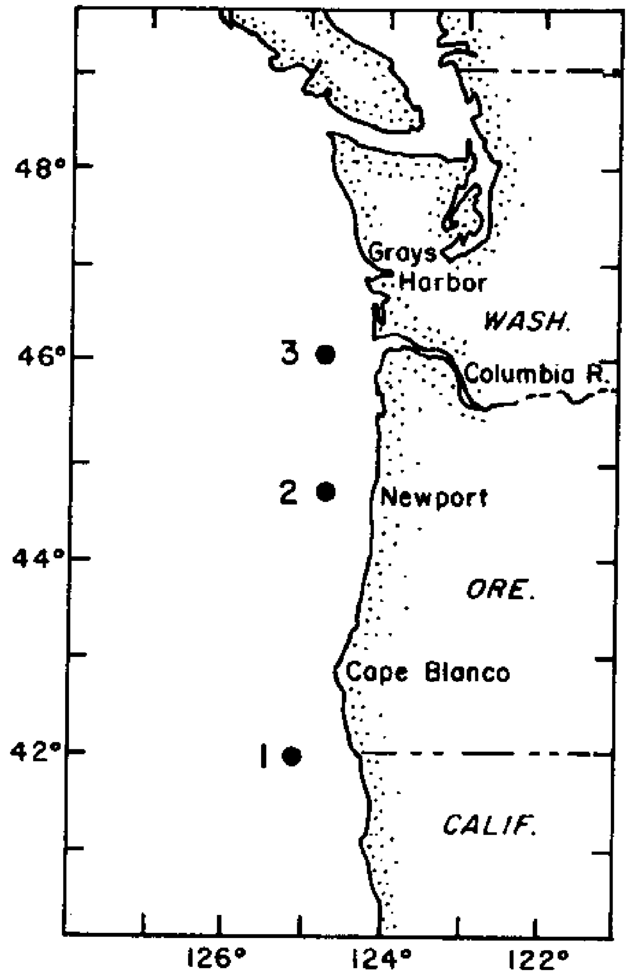


Figure 3-1. Location of deep water hindcast station (from National Marine Consultants, 17).

Station	Season	Type	N-NW			NW-W			W-SW			SW-S			Average		% Calm or Offshore
			Ho	To	%	Ho	To	%	Ho	To	%	Ho	To	%	H	T	
1	Winter	Swell	5.7	9.8	19.6	5.3	10.5	56.6	5.5	10.4	19.6	3.8	9.4	4.2	5.4	10.3	29.2
		Sea	4.1	6.9	15.0	5.3	7.1	8.1	6.5	7.6	13.7	8.2	8.1	33.8	6.6	7.6	
	Spring	Swell	2.9	9.9	15.7	4.8	10.5	65.8	2.7	10.2	13.4	3.3	9.4	5.0	4.1	10.3	26.9
		Sea	4.5	6.9	41.1	3.5	6.2	4.7	4.5	6.9	10.1	4.0	6.8	17.3	4.4	6.8	
	Summer	Swell	2.6	10.2	38.8	3.3	10.3	52.9	2.8	9.3	6.6	2.0	11.0	1.5	4.6	10.1	29.2
		Sea	4.8	6.9	59.5	3.0	5.6	2.8	3.3	5.9	3.9	3.2	5.9	4.7	4.5	6.8	
	Autumn	Swell	4.8	10.0	25.0	4.7	10.5	64.0	3.3	9.5	9.8	2.7	9.3	1.2	4.6	10.3	39.7
		Sea	4.6	6.9	27.4	4.3	6.7	5.1	4.2	6.7	9.9	5.0	7.1	18.0	4.6	7.0	
	Annual	Swell	3.7	9.9	24.8	4.7	10.4	59.4	4.2	10.0	12.7	3.3	9.7	3.1	4.4	10.2	30.3
		Sea	4.6	6.9	38.3	4.5	6.4	5.0	5.0	6.8	9.1	6.6	7.0	17.3	5.2	6.9	
2	Winter	Swell	4.5	10.0	10.1	5.4	10.7	52.2	5.2	10.2	24.7	5.6	9.4	13.0	5.3	10.3	30.1
		Sea	5.6	6.8	7.5	5.9	7.3	10.8	6.2	7.3	14.5	7.9	7.9	37.0	7.0	7.6	
	Spring	Swell	3.3	10.3	15.7	4.4	10.7	65.7	4.1	10.5	12.4	4.5	9.1	6.2	4.2	10.5	29.7
		Sea	4.0	6.3	32.8	3.9	6.2	9.6	4.6	6.6	10.4	5.3	6.8	17.4	4.4	6.4	
	Summer	Swell	2.9	9.4	33.3	3.3	10.4	59.5	3.2	8.9	4.4	3.5	8.6	2.8	3.1	10.0	29.5
		Sea	3.8	6.2	51.4	2.7	6.4	4.8	3.2	5.8	5.6	4.2	6.0	8.6	4.4	6.1	
	Autumn	Swell	4.8	10.3	17.8	5.1	10.8	68.4	3.7	10.0	7.1	3.8	9.6	6.7	3.2	10.6	35.3
		Sea	4.2	6.2	17.4	5.1	6.8	7.1	4.7	6.5	15.7	5.9	7.1	25.1	3.7	6.7	
	Annual	Swell	3.6	9.8	18.8	4.6	10.5	60.5	4.6	10.4	13.2	4.8	9.4	7.5	3.4	10.3	31.0
		Sea	4.0	6.3	30.0	4.7	6.6	7.8	5.0	6.8	10.6	6.5	7.3	20.6	4.9	6.7	
3	Winter	Swell	4.8	10.0	5.2	5.4	10.8	50.4	4.9	10.9	29.6	5.3	9.6	14.8	5.2	10.6	31.4
		Sea	5.6	7.0	5.1	7.0	7.6	11.7	6.5	7.6	13.9	8.3	8.1	37.9	7.6	7.9	
	Spring	Swell	3.3	10.0	11.5	4.3	10.8	63.7	3.8	10.3	16.3	4.1	9.1	8.5	4.1	10.5	32.1
		Sea	3.9	6.2	27.6	4.1	6.3	11.4	4.6	6.5	10.0	5.4	6.9	19.0	4.6	6.5	
	Summer	Swell	2.8	9.4	22.1	3.4	10.2	66.0	3.2	9.7	8.6	2.6	8.2	3.3	3.2	9.9	26.9
		Sea	3.6	6.0	44.3	3.0	5.6	10.5	2.9	5.6	7.4	4.6	6.6	11.0	3.6	6.0	
	Autumn	Swell	5.1	10.1	6.0	5.2	9.6	74.5	4.5	10.6	11.1	3.9	9.6	8.4	5.0	9.7	39.0
		Sea	5.0	6.8	12.8	4.9	6.8	8.0	5.8	6.9	15.8	6.3	7.2	24.6	5.7	7.0	
	Annual	Swell	3.3	9.7	11.1	4.5	10.6	62.4	4.3	10.5	17.4	4.6	9.5	9.1	4.3	10.4	31.6
		Sea	3.9	6.2	25.0	4.6	6.5	10.5	5.2	6.8	11.0	6.7	7.4	21.9	5.1	6.7	

Table 3-I Typical wave conditions off the Oregon coast (from Bourke, 26).

As shown in Table 3-II, many authorities have addressed the subject of design wave heights. Weigel[20] assumes that significant waves of the design height have occurred during the design storm over a

Authority	$\frac{H_{1/3}}{H_{av}}$	$\frac{H_{1/10}}{H_s}$	$\frac{H_{max}}{H_{1/3}}$	Source
Munk (1944)	1.53	-	-	Field data
Seiwell (1948)	1.57	-	-	Field data
Wiegel (1949)	-	1.30	1.85	Field data
Harney, Saur and Robinson	1.51(swell) 1.61(sea)	-	-	Field data
Barber (1950)	1.61	-	1.50	Theoretical
Putz (1950)	1.63	-	-	Field data
Munk & Arthur (1951)	1.49	-	1.63	Field data
Longuett-Higgins (1952)	1.60	1.27	1.77	Theoretical
Putz (1952)	1.57	1.29	1.80	Theoretical
Darbyshire (1952)	1.60	-	1.50	Field data
Hamada (et al) (1953)	1.35	-	-	Experimental
Watters (1953)	1.60	1.27	-	Field data
Yoshida (et al) (1953)	1.50	-	-	Field data
Farmer (1956)	1.61	1.24	-	Field data
Wiegel & Kukk (1957)	1.37 1.48	1.32	1.40 1.64	Field data
Darbyshire (1959)	-	-	1.45	Field data

Table 3-II Wave height statistical correlations (from Wilson, 3).

period of 3 hours with a significant period T. He has obtained the average wave period in the 3 hour interval to be T/1.24. The probable number of waves in the interval would then be:

$$N = \frac{3 \times 3600 \times 1.24}{T} = \frac{13,392}{T}$$

This value of N could be expected to approximate 1000 and justify $H_{max} = 1.86\ H_{sig}$.

During the period of the study conducted by National Marine Consultants[17], waves with significant heights in excess of 29 feet were shown to occur .04 percent of the time. The largest wave predicted by the hindcasting technique had a significant height of 31.5 feet. Application of the height relationships from Table 3-III, result in a design wave, H_{max}, on the order of 59 feet, with a period of 13 seconds.

The general conclusions as to the wave pattern to be expected in the Oregon coastal region as presented in the hindcast data are:

1. The predominant direction from which local swell approach is the NW-W octant during all seasons.

2. The predominant direction from which local seas approached was N-NW during the summer and spring and SW-S during fall and winter.

3. Waves generated by local storms were higher than swell.

4. The highest waves, irregardless of direction, occurred in winter.

5. The highest waves generally come from the SW-S octant.

6. The period of swell was always greater than local sea.

7. The shortest periods for both sea and swell occurred in summer.

8. The longest period swell occurred in autumn, the longest period sea in winter.

9. The largest swell approached from NW-W.

10. The largest sea approached from SW-W.

11. The average percent of calm sea days was 31% with autumn having the greatest at 35.5%.

No. of Waves n	H_{max}/H	No. of Waves n	H_{max}/H
20	1.25	200	1.64
50	1.42	500	1.77
100	1.53	1000	1.86

Table 3-III Height relationships of maximum and significant waves (from Wilson,19).

3.2 WINDS

The Oregon Coast is located in the center of a zone of prevailing westerlies with local winds varying from northwest to southwest throughout most of the year.

During the summer the North Pacific High reaches its greatest development and is centered about 30-40° N, 150° W. The Aleutian Low is weak during this period, and the interaction of these two pressure zones favors summer winds generally from the northwest to north over the nearshore areas.

During the winter the North Pacific High weakens and its center shifts about 10° to the south while the Aleutian Low intensifies. This results in winds, frequently of gale force, approaching from the southwest.

Duxbury et al[21] have computed geostrophic winds for the area from atmospheric charts. The velocity aloft was determined, and corrected by rotation of the wind vector 15° to the left of its downwind direction and reducing the speed 30% to obtain a surface wind applicable to a height of 10 meters above the surface of the water. The results for two stations are shown in Figure 3-2. The wind rose shown at Newport was computed from wind data over a two-year period from 1969 to 1970. Additional wind data is available for the Newport area from a wind recorder located on the south jetty, where it is relatively free of land effects.

The data shows a reversal of direction from January to July with January having wind from SW-S and July having wind from W-NW. The winds rotate back to the January conditions during the late summer and fall. The maximum winds to be expected are from the W-SW and average 10 to 11 knots during November and December recurring in February. The maximum summer winds occur in June and July from the W-NW at 7 to 8 knots on the average.

The monthly averages are deceptive, and very high winds are experienced during certain times of the year. Duxbury et al[21] have shown that during the period from December to February winds in excess of 45 knots may be expected about 5% of the time. These winds are generally from the S-SW and are more prevalent along the Southern Oregon Coast.

3.3 CURRENTS

A complex circulation pattern prevails along the Oregon Coast as a result of the interactions of the various types of currents present. This complex regime includes permanent ocean currents, local wind driven currents, currents due to upwelling and tidal currents. The overall effect of the current system on the structure of the proposed plant is probably negligible due to the weak and variable nature of the currents, but the possible effects of the nearshore flows on the plant discharge system are considered in Chapter 4.

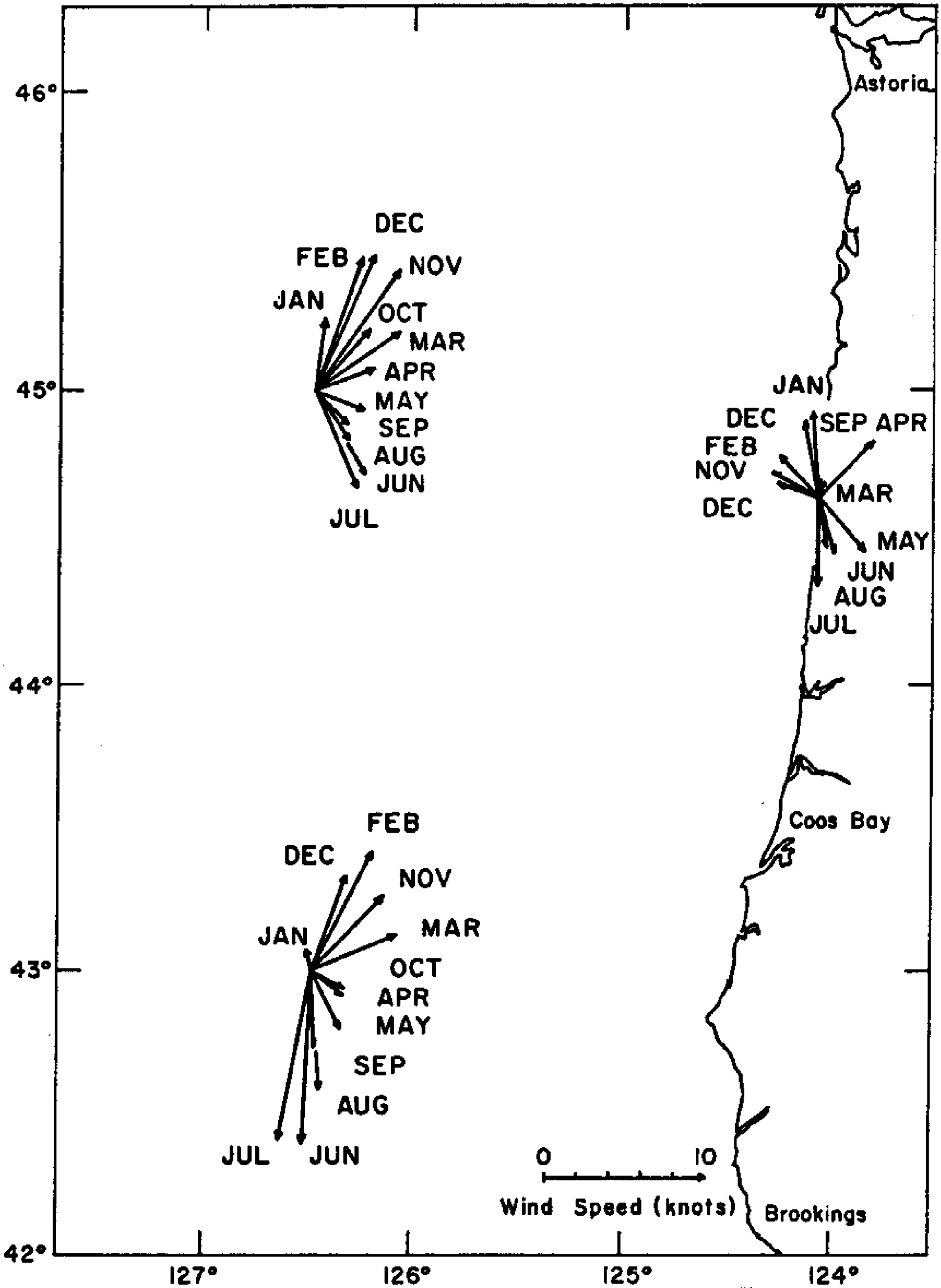


Figure 3-2. Geostrophic wind roses.

3.3.1 Permanent Currents

The permanent ocean currents off the Oregon Coast are known only in general terms. The circulation pattern has not been described in detail. In general the California Current is a broad, slow, shallow, southward-flowing current. It flows offshore and is about 300 miles wide with an average speed of 0.2 knots. Its strength varies with the wind field, reaching a maximum during the summer months when the winds are consistently from the N-NW.

The Davidson Current is a seasonal flow to the north which reaches speeds of 0.5 to 0.9 knots. This current develops off the Oregon Coast in September and becomes well established by January. The current is at least 50 miles wide. The driving force is not well understood but the current diminishes in the spring, disappearing by May. This tends to show a great deal of dependence on the local surface wind field. Sverdrup[22] has advanced the concept that the current is the surface manifestation of a deeper northward-flowing countercurrent and appears when the seasonal opposing winds weaken.

3.3.2 Wind Driven Currents

The direction of current flow has been shown to follow closely that of the prevailing winds. In the Oregon coastal zone the winds from the sea are deflected by the bluffs and the Coastal Range so that they generally follow the coastline. The storm winds of winter with their high velocities are generally southwesterly, while the prevailing winter wind has an easterly component. The summer wind, in contrast, is generally from the northwest or north.

As a result of these seasonal wind patterns, the nearshore currents are generally southward in summer and northward in winter. These currents have been found to be highly responsive to changes in wind direction.

Ekman[23] found that for a homogeneous body of water of infinite depth the current velocity is proportional to the wind stress. In the Northern Hemisphere and in an infinite ocean the direction of the current is 45° to the right of the wind. In shallow water this angle is modified according to the ratio h/d, the ratio of the water depth to Ekman's wind-affected surface layer. In shallow water h/d decreases with increasing wind speed. Observations made at Blunts Reef and Umatilla Reef Lightship (5 miles off the coast) as shown in Table 3-IV a,b, indicate that for NW-E winds the surface current is about 30° to the right of the wind direction. For SSW-W winds the same lightship current observations indicate a surface current directed 35° to the left of the wind. This tends to show that the coastal boundary has more effect than the Coriolis acceleration.

Average current (knots) due to wind	10	20	30	40	50
			Wind Velocity (mph)		
Blunts Reef	.2	.3	.4	.7	.8
Columbia River	.4	.5	.6	.8	.8
Umatilla Reef	.2	.6	.9	1.0	.9

Table 3-IVa Average speed of current due to winds of various strength (from Tidal Current Tables,25).

Wind from (in degrees)	Blunts Reef		Columbia River		Umatilla Reef	
	L	R	L	R	L	R
N		20		35		44
NNE		6		27		18
NE		10		9		34
ENE		32		29		48
E		28		17		52
ESE		7		2		38
SE	11		8			25
SSE		13	7			6
S		1	19		6	
SSW	11		44		13	
SW	18		74		32	
WSW	28		121		52	
W	60			145	77	
WNW		2		105	6	
NW		31		78		37
NNW		43		53		25

Table 3-IVb Average deviation of current to right or left of wind direction (from Tidal Current Tables,25).

3.3.3 Currents Due to Coastal Upwelling

During the summer months, June throughout September, the process of coastal upwelling occurs along the Oregon Coast. Cold, saline water upwells in eddies due to the prevailing northerly winds which create a divergence near the coast as the surface water moves offshore and is replaced from below. The seasonal pycnocline breaks to the surface forming a front 5 to 10 miles offshore. Shoreward of the front the waters take on the characteristics typical of upwelling; relatively low temperatures, high salinity, and low dissolved oxygen. Seaward of the surface front the surface temperature is generally 5° to 7° C. warmer than inshore.

Figure 3-3 shows that during the late summer a southward flow has been found in the upper 130 feet[24] and below this depth the flow was northward. The flow tended to concentrate beneath the inclined pycnocline at a depth of 300 feet (Figure 3-4). The northward flow was not consistent, but appeared to reach maximum strength when the surface flow was small. The increase of the southward surface flow (due to winds) tended to be accompanied by a decreased northward subsurface flow.

3.3.4 Tidal Currents

The tidal currents as observed by the lightships off the Oregon Coast and reported in the Tidal Current Tables[25] exhibit a weak rotary motion, turning clockwise with a 12.5 hour period. Spring and neap tides which occur bi-weekly have a significant effect on the tidal currents, increasing and decreasing the currents by about 20%, respectively.

Bourke[26] has computed an approximate tidal current by observing the time necessary for the tide to traverse the distance from the Farallon Islands to Cape Alva off the northern Washington Coast. The distance between the two spots is 628 nautical miles and the time to traverse the distance is approximately 104 minutes, thus giving a wave velocity of 610 ft./sec. The Tidal Current Tables[25] indicate that the nearshore tidal current is mostly translational all along the Pacific Coast, setting approximately 060° T on flood and 240° T on ebb. The nearshore component of the tidal wave motion is determined by taking the cosine of 60° times the N-S velocity which results in a wave traveling 060° T at a velocity of 305 ft./sec., and approaching the beach at an angle of 30°.

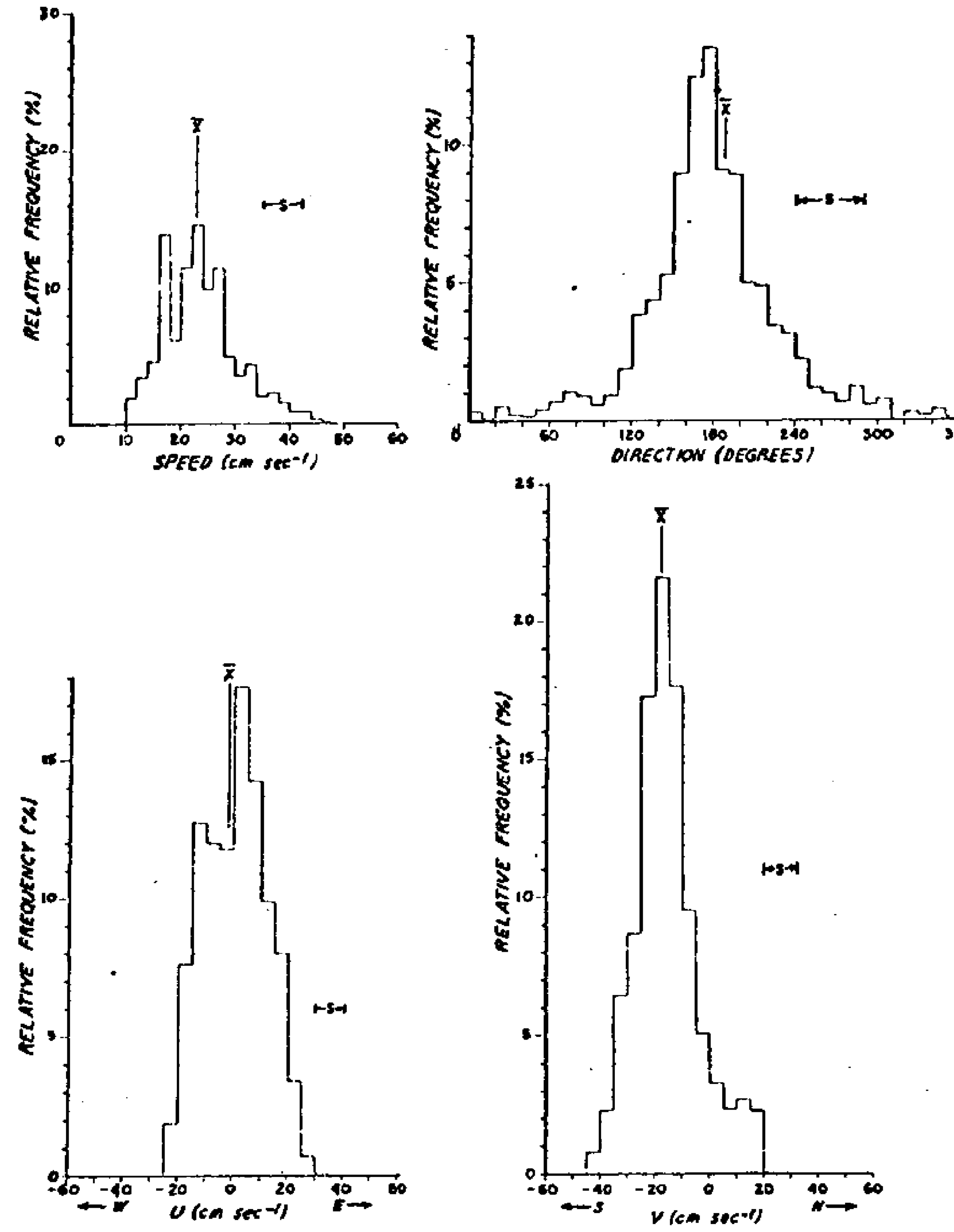


Figure 3-3. Histograms of current speed, direction, and velocity components measured 5 miles off Depoe Bay at 20 meters depth (from Mooers, et al., 24).

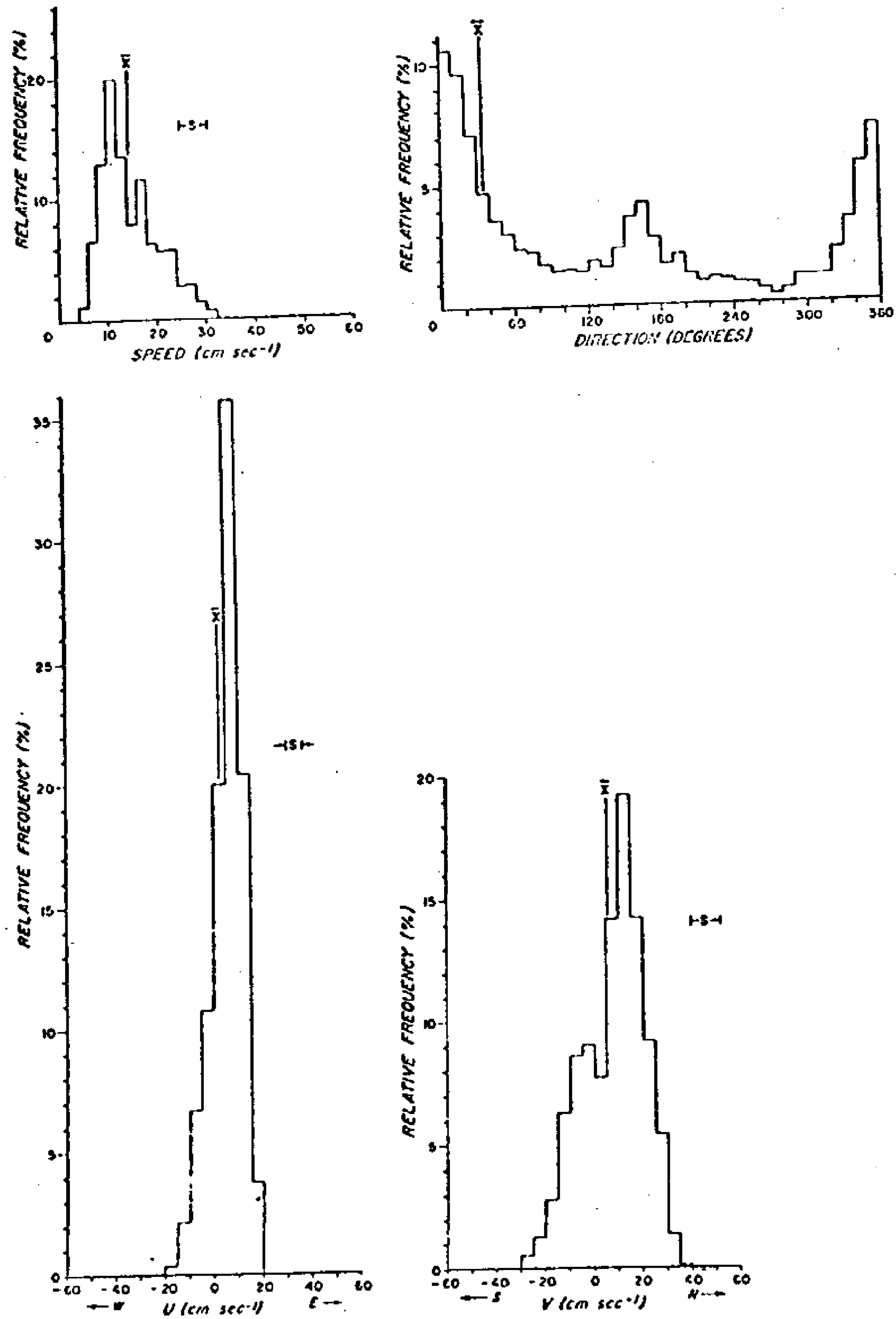


Figure 3-4. Histograms of current speed, direction, and velocity components measured 5 miles off Depoe Bay at 60 meters depth (from Mooers, et al., 24).

The maximum horizontal particle velocity, or the maximum velocity of the net tidal motion is given by:

$u = ga/c$

u = maximum horizontal particle velocity (shallow water Airy Wave Theory)

g = gravitational acceleration

a = tidal height (approximately 3.55 feet)

c = wave velocity

In this case the speed of the tidal current is approximately .37 ft./sec. This value compares favorably with the lightship measurements. Table 3-V shows values for the onshore-offshore component of the tidal current at various locations on the Pacific Coast.

3.3.5 Conclusions

Certain general conclusions may be drawn about the nearshore current regime. They include:

1. The net current within five nautical miles is generally southward in summer and northward in winter.

2. Tidal currents are translational rather than rotary due to the shallow nearshore profile. Tidal currents are small, generally near .5 ft./sec.

3. The wind-driven current frequently masks the tidal current. Wind currents nearshore are more influenced by boundary conditions than Coriolis effects. Supporting evidence may be observed by the deviation of the current to the left of the wind during southerly flow.

4. Surface currents are more pronounced near river mouths and bay entrances. These currents exhibit a semi-diurnal and seasonal fluctuation, reaching velocities exceeding 5 ft./sec. during high runoff periods.

5. The local topography greatly influences currents. This microscale structure is especially important in the vicinity of headlands and harbor jetties where combined effects of shoaling and reflection may create complex eddies.

6. During summer the southward surface flow may be underlain by a northward counterflow in deeper waters.

Location	Mean Tidal Range (ft)	Average Onshore-Offshore Component of the Tidal Current (ft/sec)
Humboldt Bay entrance	6.2	0.08
Crescent City	5.1	0.12
Coos Bay entrance	5.2	0.05
Yaquina Bay entrance	5.9	0.06
Tillamook Bay entrance	5.7	0.05
Columbia River entrance	5.6	0.13
Long Beach, Washington	6.2	0.14
Grays Harbor entrance	6.9	0.25
Pacific Beach, Washington	6.5	0.13

Table 3-V Average net tidal currents for the Pacific Northwest coastline (from Bourke, 26).

The current information presented in this study is general in nature. These are but examples of the complex flows to be expected in the nearshore regime. There is a need for detailed measurements in a given area in order to understand fully the complex flow pattern.

Current flow in this highly used zone of the marine environment must be known in detail in order to prevent or minimize impact on the environment of man's use of his marine resources.

3.4 GEOLOGY AND GEOPHYSICS

Knowledge of the composition of the earth beneath any large structure is of prime importance if during its design life, the structure is not to fail. In dealing with the high degree of reliability required in the operation and construction of nuclear power plants, the importance of a thorough knowledge of the geologic structure is increased tenfold. Before beginning construction at any site, numerous borings and seismic profiles should be obtained, soil samples taken, and a complete investigation of past seismic disturbances made. Unfortunately, it was not possible during the present study to attempt such an undertaking. However, enough information was obtained through research to give a general description of the qualities of a representative site.

3.4.1 Physiography

The offshore profile North of Heceta Head is characterized by a gently sloping sand bottom. Figure 3-5 shows the bottom profiles for various locations off the Oregon Coast. Interpolation from this figure shows that the 150 foot depth contour will lie between 2 and 3 miles from shore. Figure 3-6 shows the general geographic features off the central Oregon Coast. Perpetua Bank and Heceta Bank are roughly 30 nautical miles offshore.

3.4.2 Subsurface Characteristics

Figure 3-7 gives the general onshore geology of the Big Creek region and shows the area is compiled of volcanic basalts, flow breccias, and pyroclastic rocks. This region was chosen as a representative site for the purposes of this study. Figure 3-8 gives the composition of the upper sediments offshore from this region. From this information it may be assumed that the subsurface structure off this area consists principally of volcanic basalts, breccias, and pyroclastic rocks, covered with a layer of sand. Some information indicated that this sand layer may be from 10 to 30 feet in depth.[27]

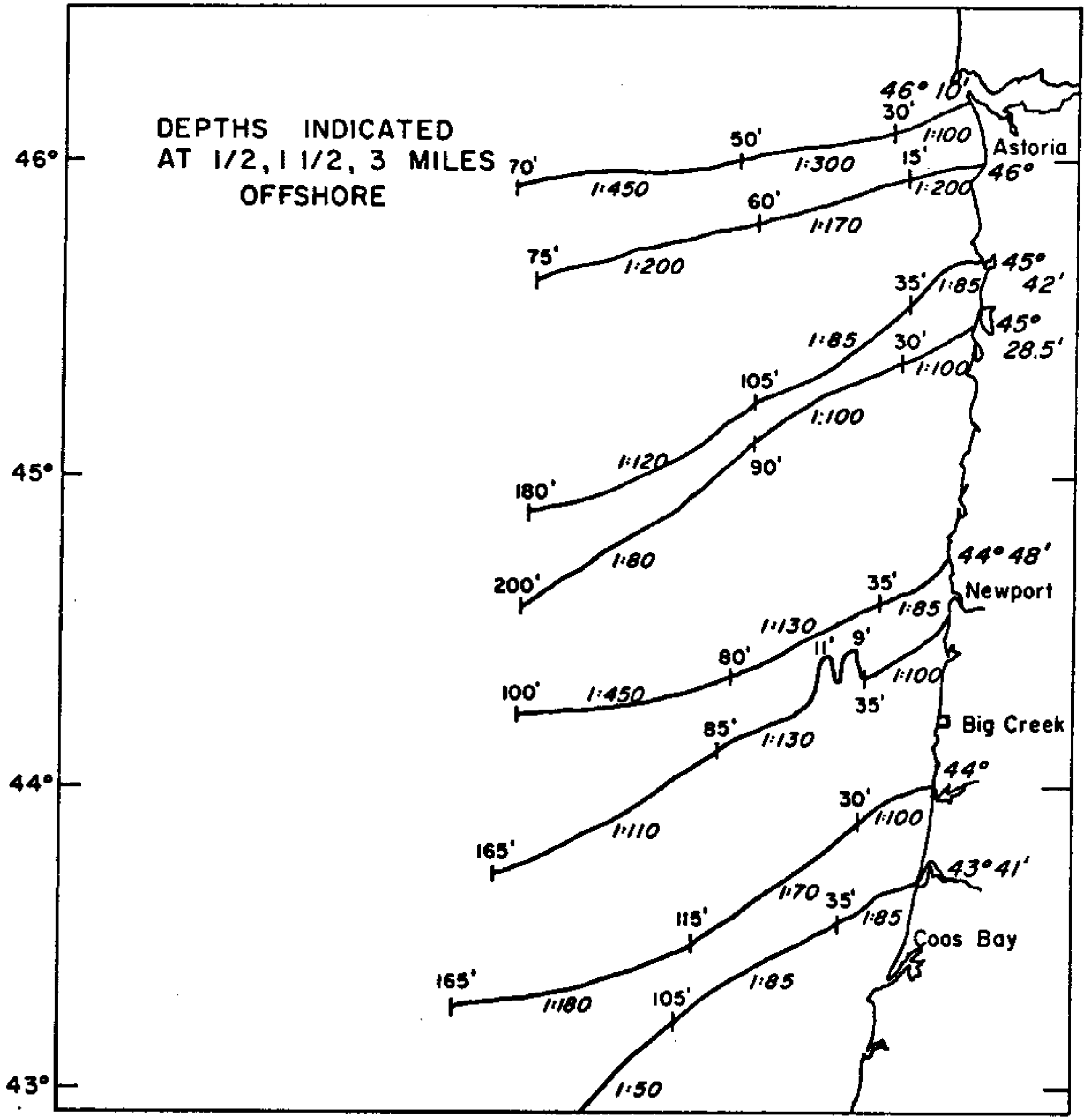


Figure 3-5. Bottom profiles (from Bourke, 26).

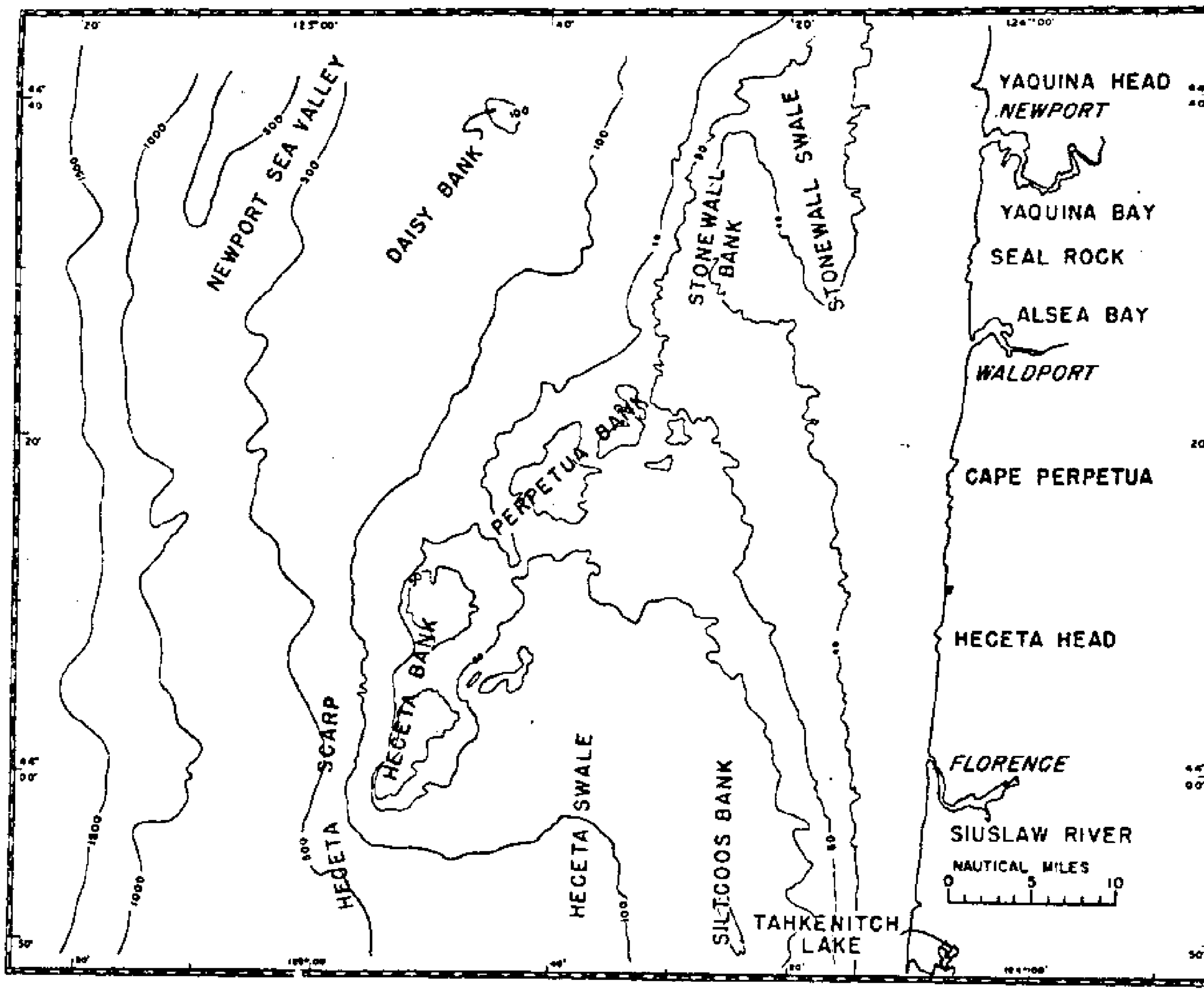


Figure 3-6. Map showing the geographic locations on the continental terrace off Central Oregon (from Maloney, 27).

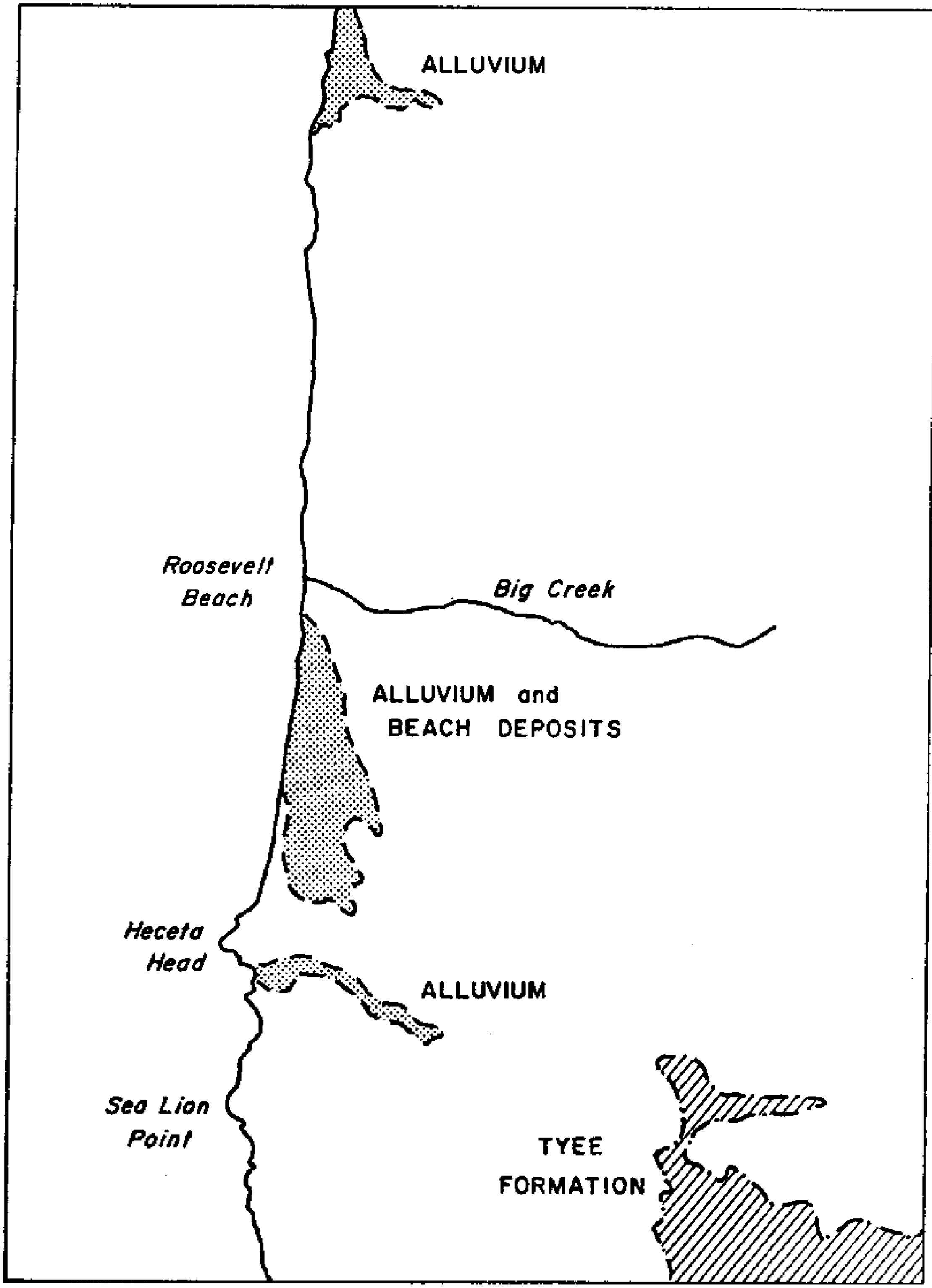


Figure 3-7. Onshore geologic formations.

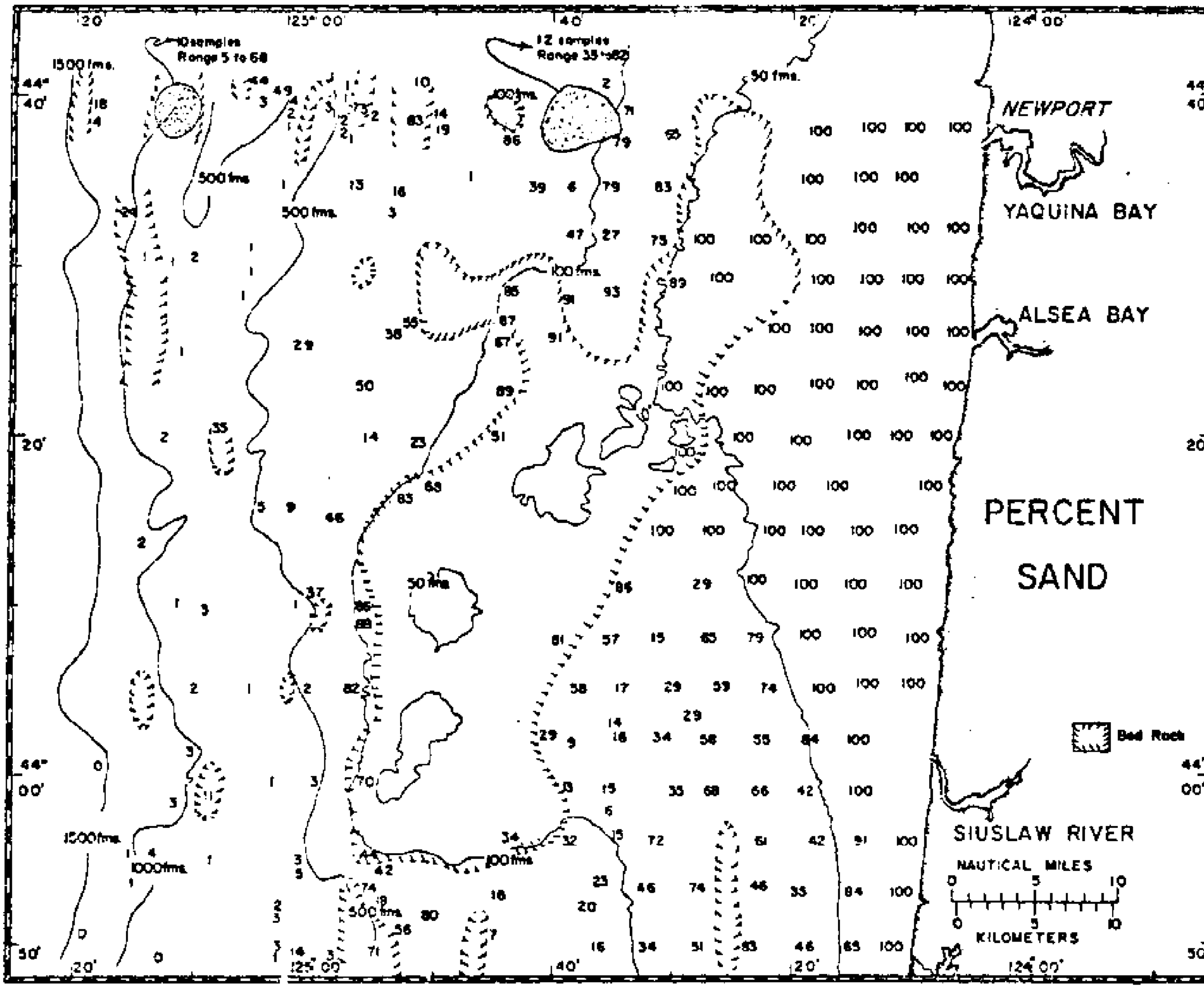


Figure 3-8. Percentage of sand in sediment samples off the Central Oregon coast (from Maloney, 27)......................

3.4.3 Seismology

A comprehensive search of seismological literature should be made to determine the location and intensity of earthquakes felt within a radius of 150 miles of the site location. This was not attempted for this location; however, it is felt that this site will have a history of low seismic activity. Figure 3-9, from studies performed for Portland General Electric's Trojan site, shows all earthquakes which were reported in northern Oregon and southern Washington from 1769 to 1969. As can be seen, the maximum reported earthquake shown is of magnitude VII (Modified Mercalli Scale). From Hershberger[28], an earthquake of intensity VII will have a maximum horizontal acceleration of .12 g. Thus, for preliminary design purposes, until a complete seismological literature search could be accomplished, a design earthquake of intensity VII would be a reasonable value.

Figure 3-10 of the geology of western Oregon, shows that faulting has occurred just south of the Big Creek area. These faults are probably minor and this location is not near any major fault zone; however, they should be thoroughly studied during site investigations. Dr. Couch of Oregon State University stated during studies performed for Trojan that there has been no confirmed historic surface faulting in Oregon. One reason for this is that the focus depth of earthquakes occurring in Oregon is unusually deep, thus reducing the chances of surface breakage.

3.5 TSUNAMIS

In any offshore study conducted for the Pacific Basin, the possibility of a Tsunami occurring during the life of the structure is a factor which must be considered. The pre-design plan usually consists of a survey of past Tsunami producing earthquakes, and the results in the area under consideration. The example used as a base in this study is the Alaskan earthquake which occurred on March 27, 1964. Wilson[29] estimated that the probable time between occurrences of earthquakes in the Alaska-Aleutian Island chain, with magnitudes of 8.5 on the Richter scale, is thirty years. Since many nuclear power plants have a design life expectancy of thirty years, it would appear that this earthquake would be a good example of what could be expected to occur, since the earthquake in this study registered 8.6 on the Richter scale.

Studies of past Tsunami heights (H) and periods (T) as functions of earthquake magnitude (M) have led to the empirical relationships:

$$\log T = (5/8)M - 3.31$$

$$\log H = (3/4)M - 5.00$$

These relationships will give one an idea of the wave height and period to be expected within 500 miles of the epicenter.

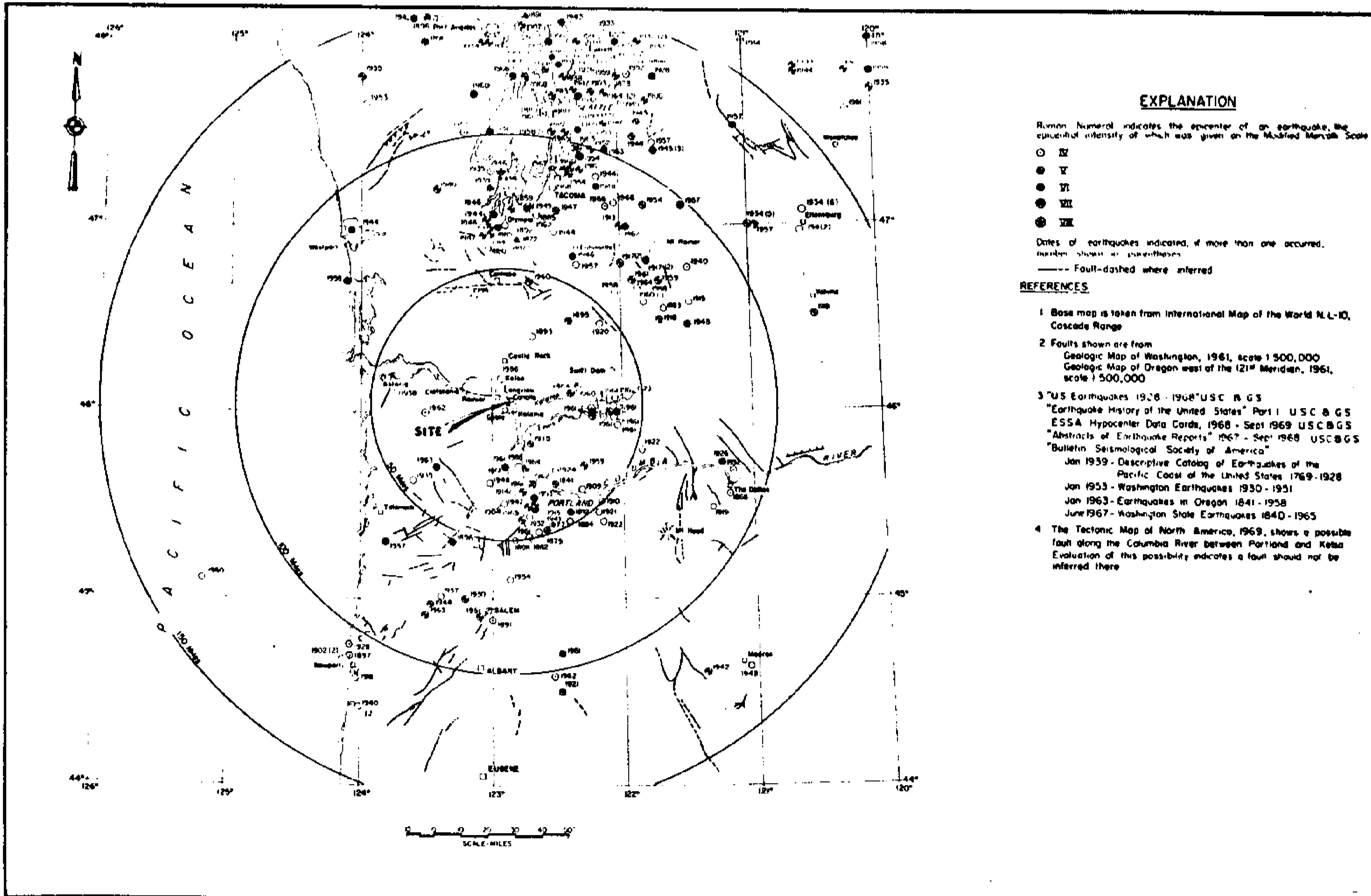


Figure 3-9. Significant earthquake epicenters and intensities at

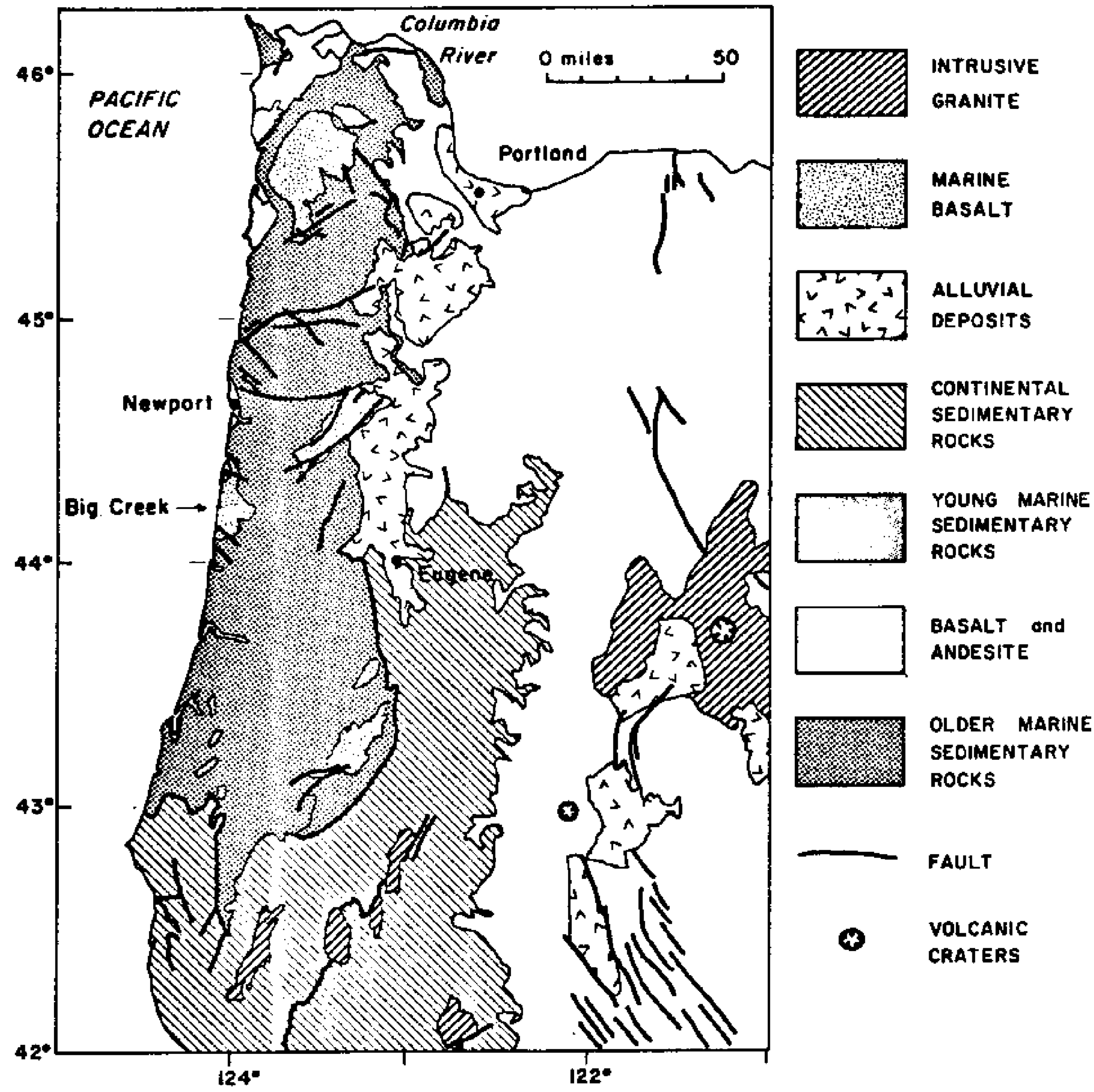


Figure 3-10. Geologic map of Western Oregon...

An analysis of the data compiled on the Alaska earthquake shows a mean period of 1.72 hours as measured at various stations along the Pacific Coast of North America. Figure 3-11 shows the heights measured at various points along the coast of Oregon. The height at Astoria was measured at 10.5 feet above mean high water (Figure 3-12). This value was obtained after the wave had entered the Columbia River, thus it is in relatively shallow water, and larger than would be expected at offshore depths.

Assuming the Tsunami follows shallow wave theory, the wave speed at a water depth of 150 feet is 67.1 fps., and the wave length is found to be 6925 feet. The most important aspect of the Tsunami to be considered in an offshore design is the force exerted by the wave on the structure. Wilson [29] has done an analysis of the effects of the Tsunami surge on large structures which might deter the advance of the wave. He has defined the force F per unit length of wall as:

$$F = 1/2\ \rho g d_w + C_f \rho d_t u_s^2 \qquad \text{where } u_s = 2gd_s$$

Figure 3-13 is a schematic diagram showing the terms as used in the equation. The depth of water formed at the wall is represented by d_w, d_t is the depth of water at the toe of the wall before the deflection of the stream, u_s is the surge velocity appropriate to the surge height d_s, ρ is the mass density of sea water, and C_f is a dimensionless constant or force coefficient, depending on the slope of the water surface of the wedge or the angle ϕ such that for $\phi = 0$, $C_f = 1$; and for $\phi = 60°$, $C_f = 3$.

Wilson [29] showed that the effective height of runup, d_w, on the wall is a linear function of $u_s^2/2\ g$ and for a fairly flat surge such that $d_t = d_s$, $C_f = 1$, the equation may be simplified to the form:

$$F = C_p\ (1/2\ \rho\ g d_s^2)$$

where C_p is a pressure coefficient representing the number of times the static pressure is exceeded. This coefficient has been determined experimentally to be about 14. This simplifies the equation further to the form:

$$F = 7\ \rho\ g d_s^2$$

This is the force per unit length exerted by the Tsunami surge.

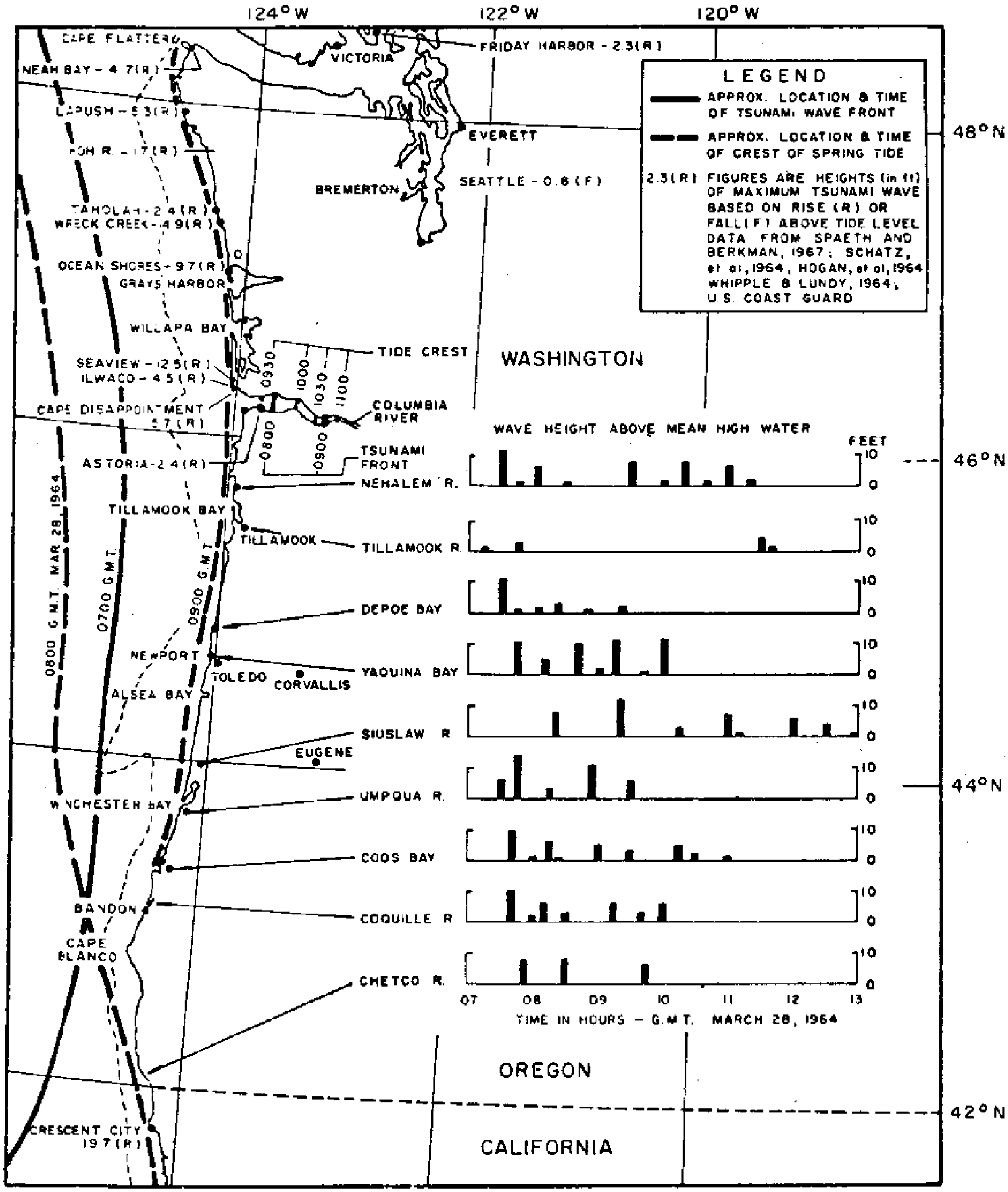


Figure 3-11. Record of wave heights at coastal stations due to the 1964 Alaskan earthquake (from Wilson and Tørum, 29)..

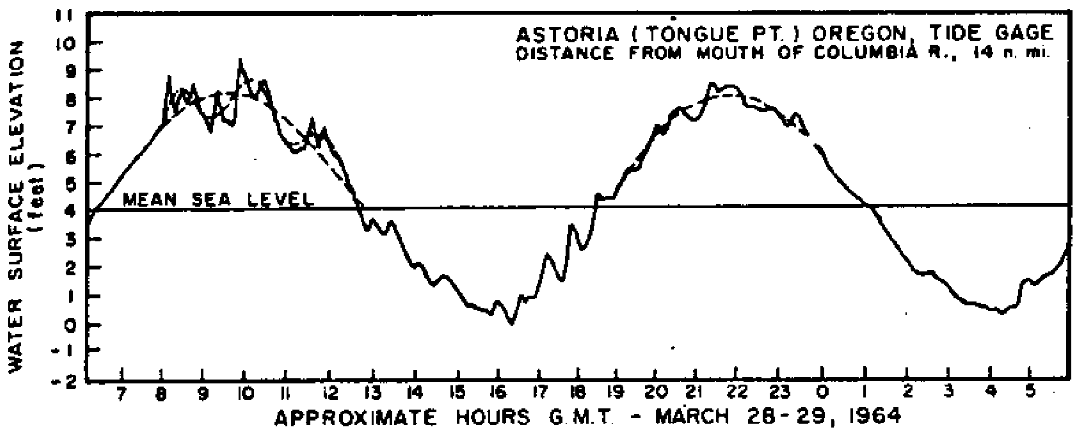


Figure 3-12. Maximum heights of tsunami waves recorded at tide stations or by observation along the Washington-Oregon coastline (from Wilson and Tørum, 29).

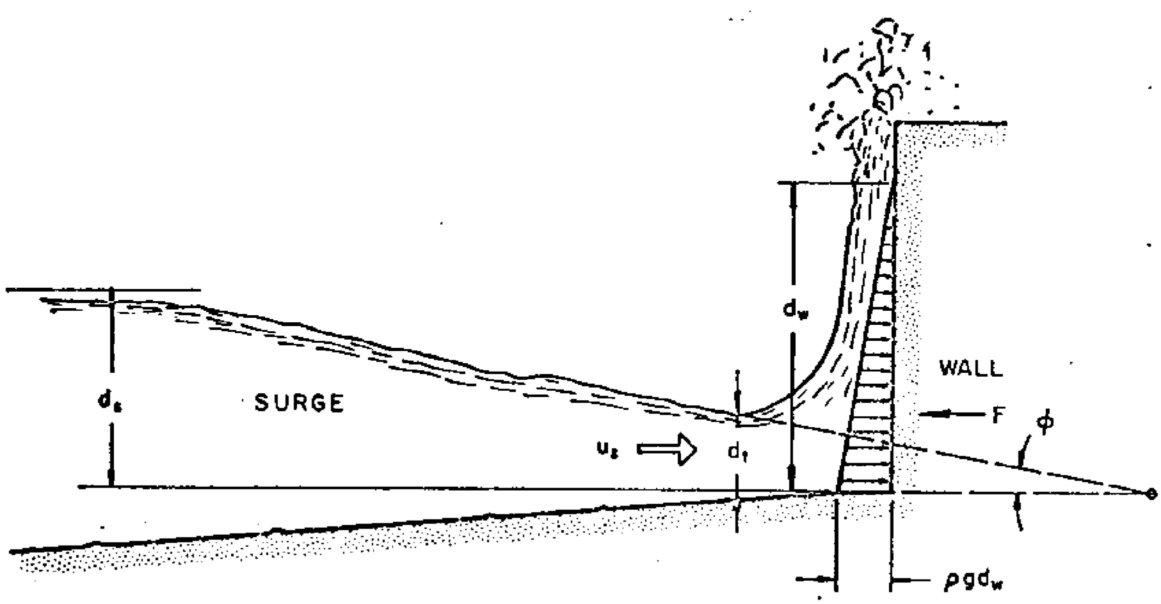


Figure 3-13. Schematic diagram of impulsive force of a tsunami surge on a wall (from Wilson and Tørum, 29)..........

3.6 BIOLOGICAL/CHEMICAL OCEANOGRAPHY

It is universally recognized that temperature affects aquatic life more profoundly than perhaps any other single environmental factor. Unfavorable temperature may affect reproduction, metabolic rates, growth, larval survival, and all of the processes of life normal to a healthy organism. Generally the proposal of a nuclear power generating plant raises a host of biological questions as to the possible deleterious or beneficial effects to the aquatic life of the heated waters resulting from reactor cooling. It is unreasonable to assume that any person in a position of responsibility will recommend the uncontrolled release of heated water to the environment. Standards are being set and enforced, and it has been shown in previous studies that catastrophic kills are not the common occurrence that some over zealous alarmists would lead us to believe.

3.6.1 Biological Considerations

In order to predict the effects of thermal discharges on an aquatic community it is necessary to have certain information about the natural environmental regime and the thermal responses of the organisms involved. Although several studies have been made of the thermal effects of heated discharges from various nuclear power sites, these studies have limited application due to:

1) The regional variation in chemical, biological, and physical characteristics of the various receiving bodies of water.

2) The difficulty of applying laboratory studies to actual situations which are less than ideal in comparison.

3) The unknown effects of interraction between the various environmental factors in addition to temperature.

4) The variance in biota encountered, and the differences in adaptation or adaptability among or within the species to be encountered.

The design of a nuclear plant should be such as to diminish the effects of heated effluents on the environment. The location of the plant in a zone which is not so sensitive to small changes of temperature as might be encountered in an estuary or river is highly desirable. The potential for dilution and rapid dispersal of the effluent in the sea is self evident.

Much of the discussion of the effects of thermal effluents centers around the immediate lethal effects on adult organisms which usually occur in the immediate vicinity of outfalls. Such effects would have little importance from an ecological or population-biology view if they were contained within narrowly restricted localities.[30] The containment of any potential kill of adult organisms within the immediate vicinity of the plant is one of the primary assets of a location offshore. The zone of higher than normal temperatures will be minimized in this proposed environment.

Since any probable site for an offshore nuclear plant is within the plankton rich upwelling zone of the Oregon Coast, this is a critical factor which must be studied in depth prior to erection of such a plant. At Hanford many plankton samples were taken, above and below the reactor plumes, and no apparent difference was found between samples as to quantity of planktonic organisms. Although biologists are aware of the potential harm to plankton passed through condenser cooling water, little work is being done in this problem area, although some work is being done at operating nuclear power plants. At Indian Point, New York, the intake water and discharge water were sampled to determine possible damage to phytoplankton. The conclusion was that there was no significant effect.[31]

A quantitative study of the zooplankton in the intake and outfall of a power plant was undertaken by Markowski in 1959.[32] All of the animal phyla found in the intake waters were likewise found in the effluent, often in greater individual abundance. He concluded that passage of cooling water through power plant condensers had no harmful effects upon the organisms. Those exposed to chlorination (.2 mg/l residual), turbulence and heat were not only alive, but able to reproduce.

The significance of phytoplankton killed by a nuclear power plant may be difficult and expensive to assess in terms of the impact of the kill on the entire phytoplankton population. After all, many pelagic fish larvae suffer greater than 99.9% natural mortality. Any studies should be carefully directed, especially in the light of the effort and expense involved in an exhaustive study.[33]

3.6.2 Chemical Considerations

The chemical aspects generally raised are those of the effects of the heated water effluent on the water quality. Monthly measurements of intake temperature, discharge temperature, dissolved oxygen, pH, turbidity, 5 day BOD (Biochemical Oxygen Demand), oil, and grease are taken at many thermal plants in California as part of the monitoring programs of the State Regional Water Quality Control Boards. None of these parameters, with the exception of temperature, has been changed as a result of thermal discharges.[34]

Many times the charge is heard that the discharge of heated effluents decreases the oxygen carrying ability of the water. This criticism is based upon laboratory conditions which point out a decrease in the dissolved oxygen capacity of heated water from 100% saturation. Actual field evidence indicates that the passage of cooling waters through the condensers does not cause a decrease in the dissolved oxygen content. The water merely becomes supersaturated at the higher temperature, and after leaving the plant the temperature decreases and the saturation level also decreases while the dissolved oxygen content remains the same.[34]

4. Conceptual Power Plant Design

The plant considered in this study utilizes a High Temperature Gas Cooled Reactor (HTGR) and one high pressure steam turbine-generator to produce 1,100 megawatts of electrical power. Miscellaneous reactor service systems and site auxiliary equipment support the operation and refueling of the plant. The entire system is contained in a "hull" consisting of four independent prestressed concrete modules: Reactor module; Turbine-generator module; Reactor service module; Site auxiliaries module. These modules are in turn supported by a large (600' x 450' x 50') prestressed concrete barge.

This section of the report includes a typical plant layout and brief description of the major plant components, a discussion of barge and hull construction techniques and corrosion considerations, stability predictions, and possible design configurations for the sea water cooling and electrical power transmission sub-systems. There are many alternative designs; this is just one example.

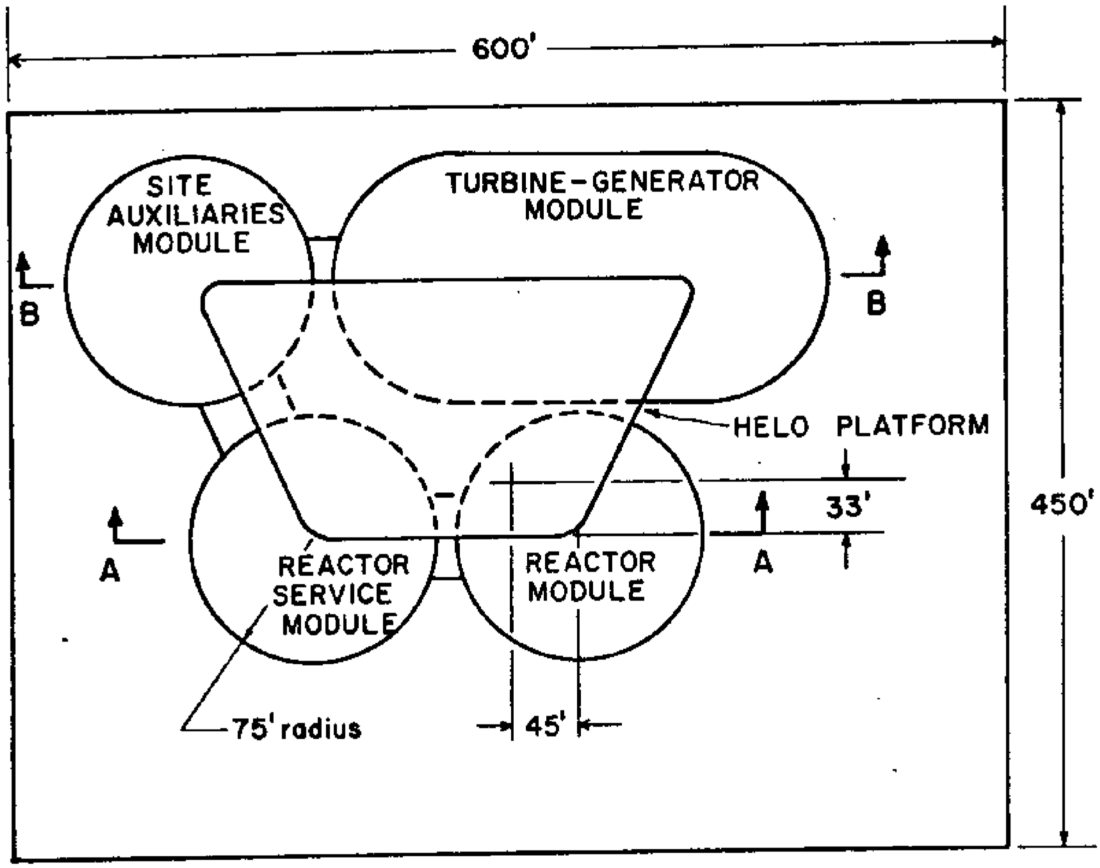
4.1 PLANT LAYOUT AND COMPONENT DESCRIPTION

The overall plant layout and configuration is depicted in Figure 4-1. Each of the prestressed concrete modules is a completely enclosed, quasi-independent structure. Three of the modules are right circular cylinders capped with spherical domes, and the fourth module is simply another cylinder that has been halved and extended 150' in the middle. Interconnecting tubes provide access for personnel and a means of linking together various related sub-systems in different modules (e.g. steam and feedwater pipes between Reactor module and Turbine-generator module). The asymetric placement of the modules on the barge is a result of the disproportionately large percentage of the total plant weight (approx. 177,000 tons) contributed by the Reactor module (approx. 92,000 tons). The hull is recessed into the prestressed concrete barge and the two are structurally integrated. A large platform atop the domes provides a helicopter platform as well as a secondary means of transportation between modules.

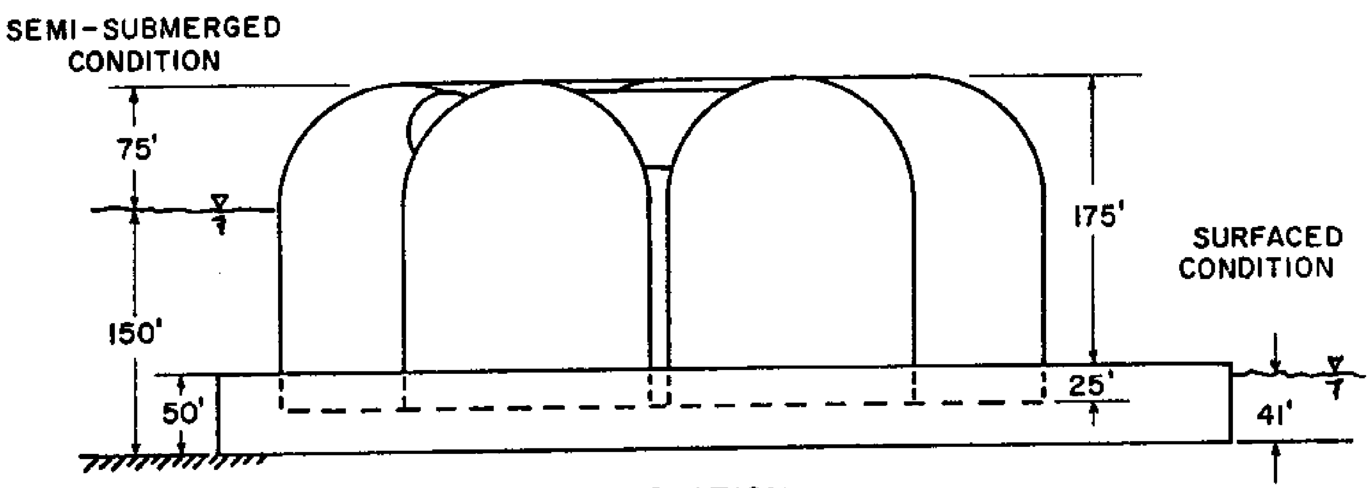
Since the major components in the system are already conveniently grouped by being contained within a specific hull module, each module and its major components will be described. (See Figure 4-2)

4.1.1 Reactor Module

The reactor module is the heart of the plant, since it contains the entire HTGR system. With the exception of the fuel handling facilities located above the core, the reactor is the only component in this module.

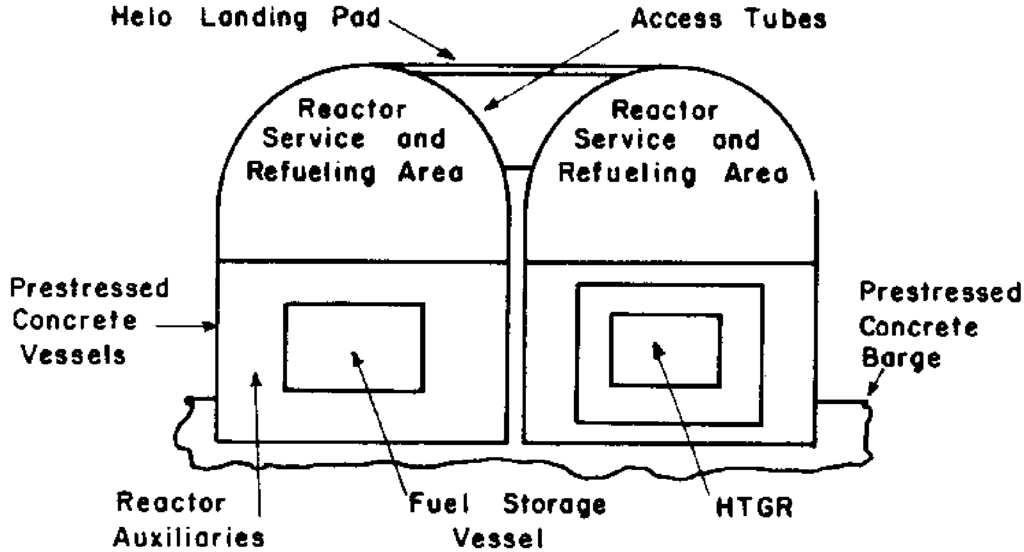


PLAN VIEW

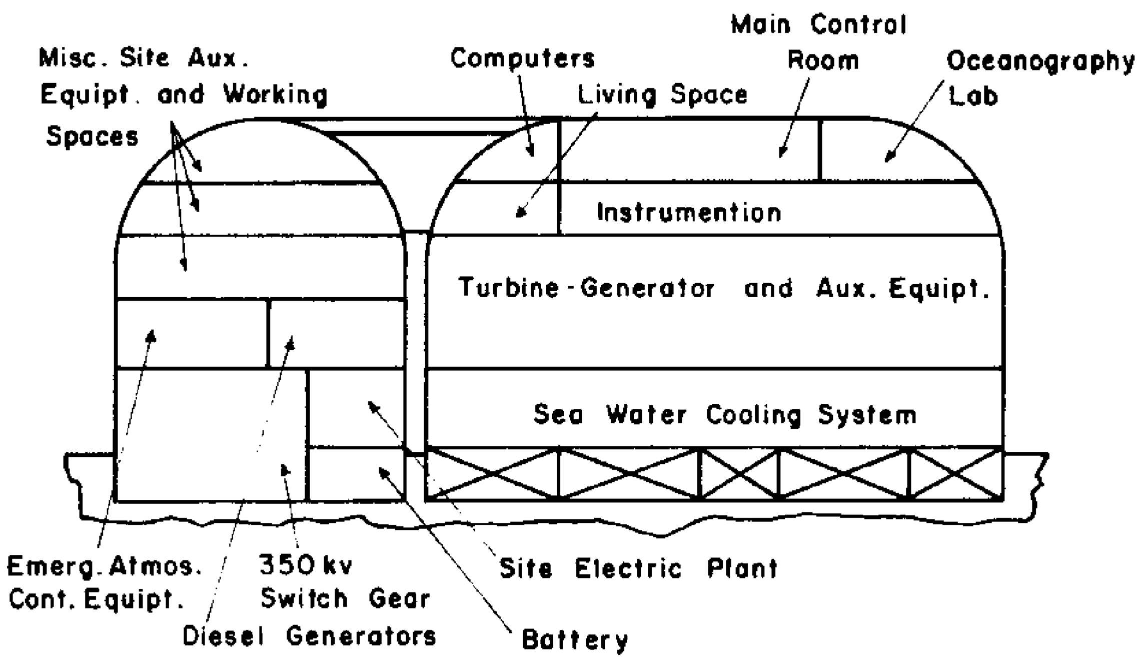


SIDE ELEVATION

Figure 4-1. Plant layout and configuration



Section A-A



Section B-B

Figure 4-2. Plant crossections.

All integral parts of the primary system, including the steam generators and helium circulators, are housed in a steel-lined, Prestressed Concrete Reactor Vessel (PCRV). In addition to housing the reactor plant, the PCRV provides the required biological shielding. Housing of the complete primary system in the PCRV permits a compact installation and eliminates major external primary coolant piping.

The HTGR uses helium gas as the reactor coolant and graphite as the moderator and core structural material. The fuel is a mixture of highly enriched uranium and thorium, used in the form of carbide particles individually clad with ceramic coatings. Several of the advantageous features of the HTGR which led to its selection in lieu of a conventional Pressurized Water Reactor (PWR) are:

1. Because of the high operating temperatures achievable with a gas coolant, the HTGR has a higher thermal efficiency (about 40%) than a typical PWR (about 30%). This means that less waste heat is rejected to the environment.

2. The HTGR minimizes the amounts of liquid, solid, and gaseous wastes that must be handled or disposed of, and essentially none of it must be released directly to the environment.

3. The HTGR requires a smaller loading of natural uranium in the core than a PWR, since additional U^{233} is bred from the thorium during reactor operation. This also provides increased latitude in the refueling schedule.

4. Containment of the entire primary system in the PCRV permits access to the Reactor module during plant operation.

5. The highly superheated steam produced by the HTGR permits use of a standard 3600 rpm turbine-generator.

6. The HTGR also has several inherant safety features, such as a large heat capacity core, moderate pressure (700 psig) primary system, atmospheric pressure decay heat removal capability, immunity of the PCRV to catastrophic failure, etc.[35]

4.1.2 Turbine-Generator Module

This module is 300' long to accomodate the massive 1,100 MWe steam turbine-generator unit. In addition to the steam plant, the sea water cooling system and related auxiliary systems, the majority of the control and instrumentation equipment is contained in this module.

The turbine-generator is a standard 3600 rpm unit that utilizes 2400 psig, 950/1000° F superheated steam from the Nuclear Steam Supply (NSS) system. The output voltage of the generator is 18,000 volts (18KV). Figure 4-3 illustrates a typical plant flow diagram.

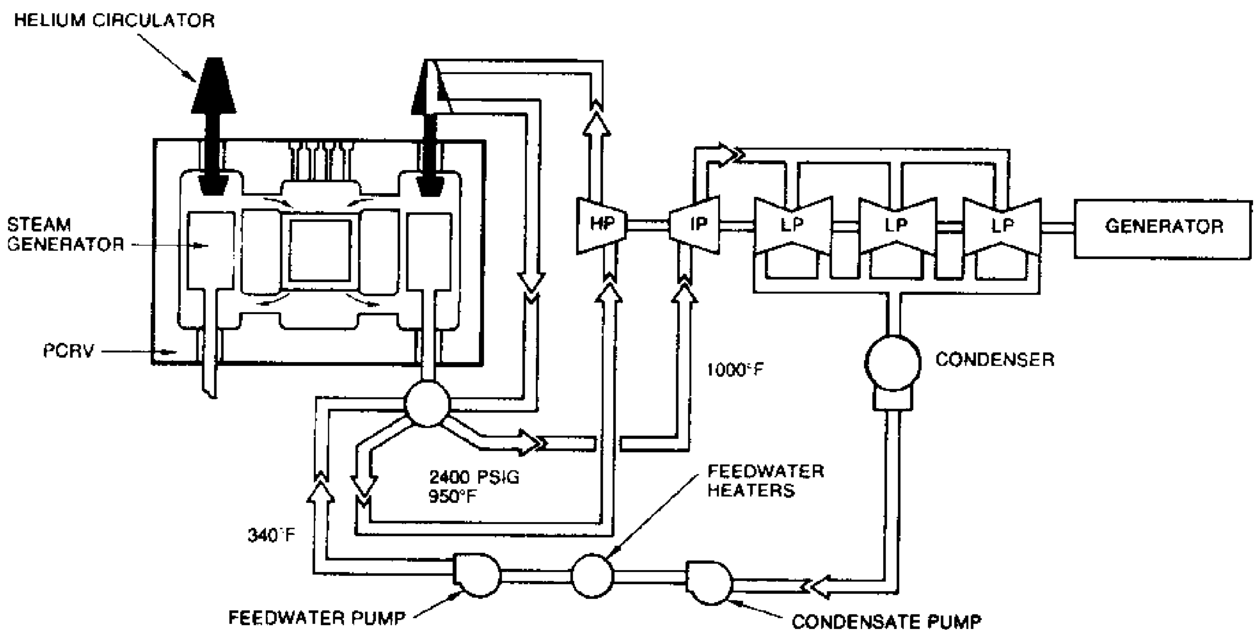


Figure 4-3. Typical HTGR plant flow diagram (from Gulf General Atomic, 35).

Another major component in the turbine-generator module is the sea water cooling system, which consists of two large intake pipes, several high capacity circulating pumps, a flow reversing chamber, and two discharge pipes. During normal operation, cooling water is taken in at the seaward end of the barge, circulated through the main condensers and discharged at the opposite end of the barge (600' lateral separation to preclude recirculation). Periodically, the direction of flow through the intake and discharge pipes is reversed for about four minutes by repositioning the gates in the flow reversing chamber. The resulting elevated temperatures in the intake pipes (about 100° F) is sufficient to kill all mussels and other marine organisms that tend to foul the pipes.[36] Figure 4-4 illustrates this cooling system configuration.

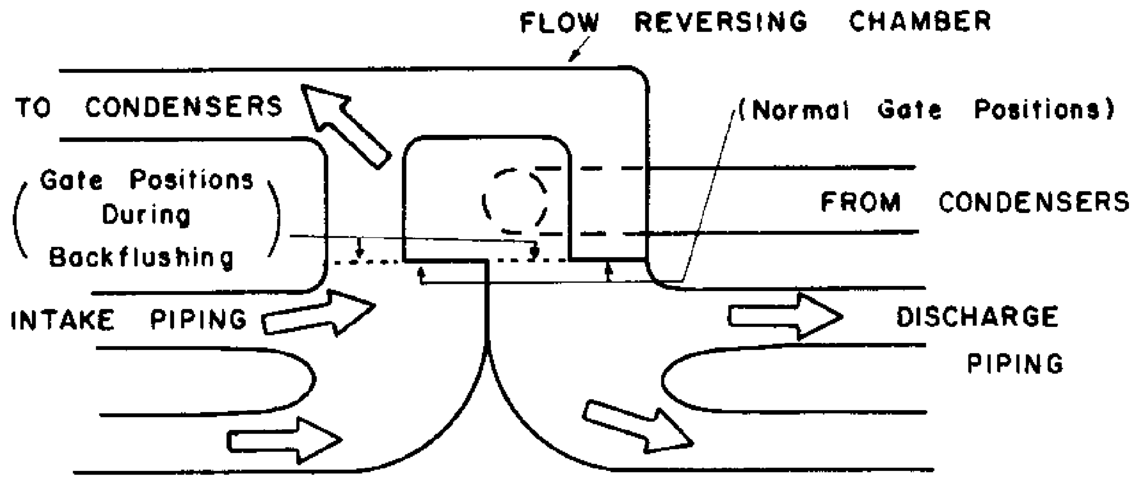


Figure 4-4. Flow reversing scheme (from Chadwick, 36).

The cooling system is discussed in greater detail in a later section.

The remainder of the mechanical systems that support the operation of the turbine-generator are also located in this module. This includes the condensate system, the feedwater system, lubricating oil systems, miscellaneous auxiliary sea water cooling systems, a distilling plant for steam plant make-up and potable water, an auxiliary fossil fuel boiler for start-up steam, etc.

This module also contains all major control and instrumentation systems for the reactor plant, the steam plant, and the electric plant, as well as other spaces not directly related to the turbine-generator. This might include items such as a computer center, a crew living area, recreational facilities, a communications room, an oceanographic laboratory (for lease to scientific institutions), etc.

4.1.3 Reactor Service Module

This module would contain the fuel storage vessel and miscellaneous primary plant auxiliary systems. The fuel storage vessel is used to store replacement fuel cells between scheduled refuelings. During refueling, the spent fuel would be off-loaded to a barge and taken to the shore facility for further shipment to a fuel reprocessing facility, and new fuel transferred from the storage vessel to the reactor core. The access tube connecting the Reactor module with the Reactor service module would be configured to provide a continuous track for a 100 ton crane between the two modules.

In addition to the fuel storage vessel, this module would house reactor support systems such as a radiochemistry laboratory, radiological control monitoring areas, a central decontamination station, the decay heat removal cooling system, a coolant sampling station, helium storage facilities, etc.

4.1.4 Site Auxiliaries Module

This module would contain all of the systems and facilities that are required to keep the site itself operating efficiently, as well as the substantial amount of switchgear necessary for transmission of the electrical power ashore.

One of the most important of these support systems is the site electric plant. The site 'hotel load' would be approximately 16,000 kilowatts, and reactor safety considerations require multiple independent sources for this power.[6] The primary source of site power would probably be a line (approx. 33 KV) from the shore substation. This supply would be paralleled by a line from the site-generated system. Two 4,000 KW diesel generators and a large storage battery would provide emergency power supplies. (See Figure 4-5 for a simplified schematic of a possible site electric plant configuration).

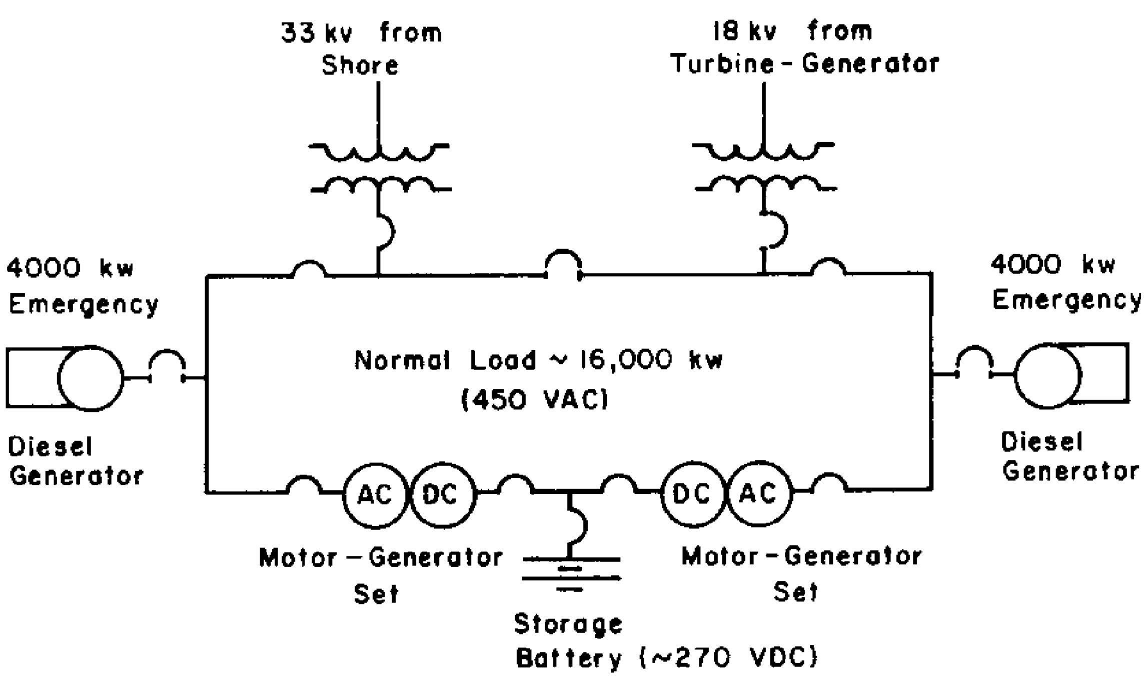


Figure 4-5. Site electric plant schematic.

A fully enclosed structure such as the power plant hull would undoubtedly be equipped with a fully automated atmosphere control system. This would include a high capacity ventilation system, a large central air conditioning system (possibly a lithium bromide absorption plant), and a heating system (possibly electric space heaters). In addition, since the hull might be completely sealed during violent storms or reactor emergency situations, it would probably be advisable to install emergency air revitalization equipment (electrolytic oxygen generators, carbon dioxide scrubbers, carbon monoxide burners, electrostatic precipitators, activated charcoal filters, etc.)

This module would also provide space for any remaining miscellaneous site auxiliaries, such as hydraulic systems, compressed air systems, sewage treatment and disposal facility, machine shops, electronic equipment repair shops, calibration shops, office space, storage space, etc.

4.2 BARGE AND HULL DESIGN

The barge and hull together form one large prestressed structure. The rectangular barge shape was a compromise between restrictions imposed by towing in inland waters and the uneven distribution of the weight of the plant. The final dimensions (600' x 450' x 50')resulted in a surfaced draft of approximately 40', with the heaviest portion of the plant (reactor module) located near the center of the barge.

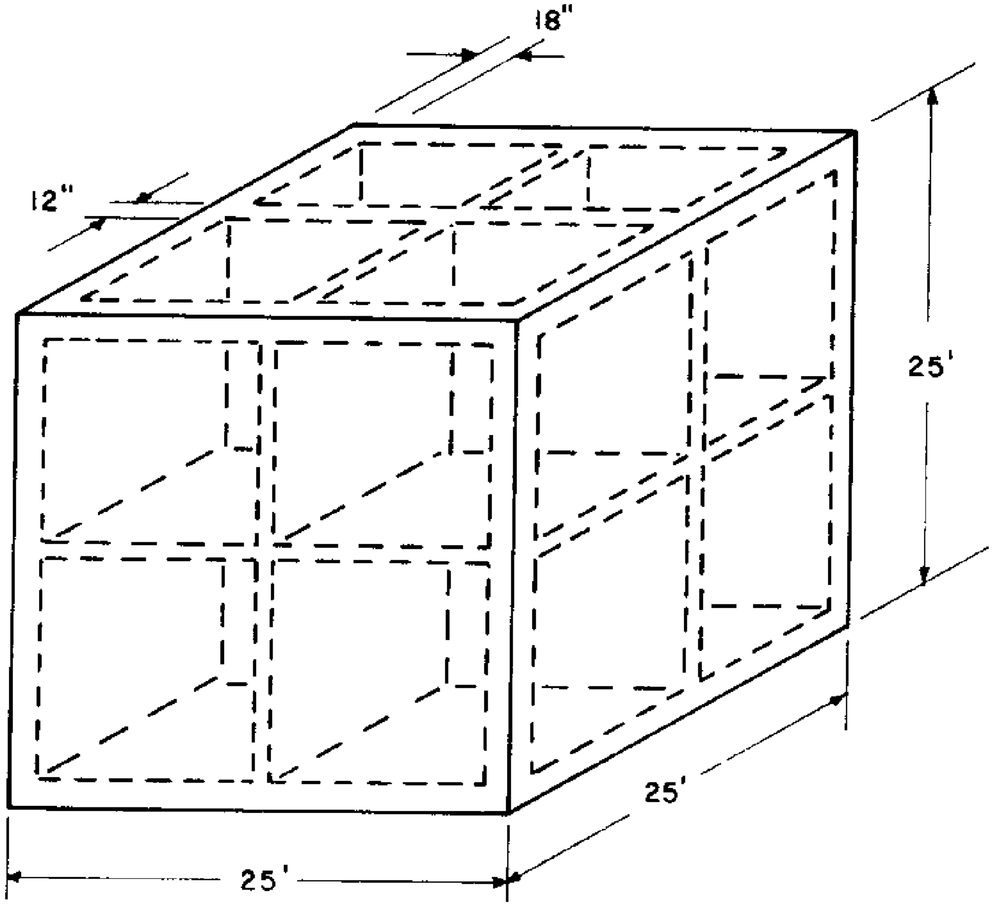
Prestressed concrete was selected as the material for both the barge and hull because it combines high strength, light weight, and corrosion resistance and is relatively easy to work with.

4.2.1 Barge

The barge could be built in a large graving dock by assembling prefabricated cubes 25' on a side. After additional reinforcement of the area under the plant, the major plant components could be landed on the nearly completed barge and the hull constructed in place. The majority of the cubes would be air-filled flotation chambers, and each would be connected to a centrally controlled venting/pressurizing system to facilitate submerging at the site. The remainder of the cubes could be used for trim tanks, for storage or fresh water/fuel, etc. Figure 4-6 illustrates a typical barge cube and a cross section of the assembled barge.

4.2.2 Hull

The hull shape is a compromise between plant space requirements, reactor safety and flooding considerations, ease of construction, and wind/wave resistance. The space requirements are estimated based on design data for Gulf General Atomic's 1,100 MWe HTGR.[6] The modular



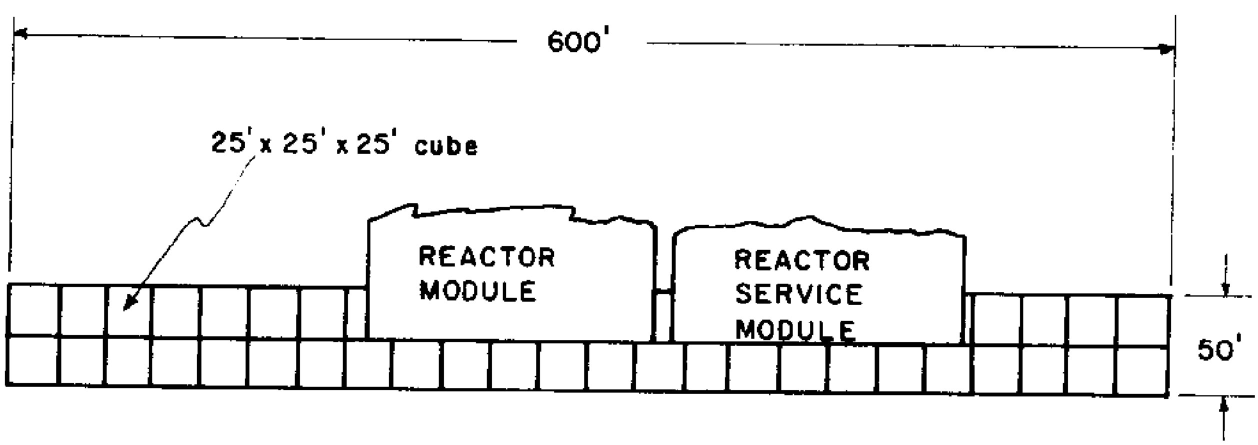


Figure 4-6. Typical pre-fabricated barge cube and assembled barge crossection.

concept was selected for several reasons. First, for safety reasons, the primary system must be housed in a separate containment structure regardless of the type or location of the plant. Secondly, the other components of the overall system lent themselves to convenient grouping within the other three modules. Thirdly, the particular hull shape was chosen based on the fact that the two most well-defined systems of thin shells commonly used in prestressed concrete construction are the dome and the segment of a cylinder.[37] Finally, the independent nature of the modules tends to minimize the severity of flooding caused by a single hull leak. The choice of a domed cylinder for a basic unit also presents a minimum effective cross section to wind and waves.

The hull cylinders would utilize circular and longitudinal prestressed reinforcing, with a wall thickness of approximately 5 feet. A spherical dome was selected for this design, but the possibility of using an eliptical dome should be considered.

4.3 SURFACED AND SUBMERGED STABILITY

The calculation of the stability of the barge and hull design was undertaken in order to assess the feasibility of transportation to the site, and placement on the foundation by selective flooding of hull compartments. It was necessary to determine if the design would maintain a positively stable state during submerging in order to predict the actions and equipment necessary for placement upon arrival at the selected site.

4.3.1 Physical Characteristics

Dry Weights

Steam Plant	35,600
Site Auxiliary	17,200
HTGR	92,200
Reactor Service Unit	32,200
Barge	150,000
Diesel Fuel/Water	20,000
	347,200 tons

Barge Draft (h)

$$W = \Delta = \rho V_{barge} = (450)(64)(h)$$

$$h = \frac{(347,200)(2000)}{(450)(600)(64)} = 40.25 \text{ ft.}$$

Submerged Displacement (d = 150')

$$\text{Barge} = \frac{(450)(600)(50)(64)}{(2000)} = 430{,}000 \text{ tons}$$

$$\text{Plant} = \frac{[(4)(3.14)(75)^2(100) + (150)^2(100)]\ 64}{2000} = 226{,}000 \text{ tons}$$

Total = 656,000 tons

Ballast Required (for submerged W = 35,000 tons)

Ballast = 656,000 - 347,200 + 35,000 = 343,800 tons

Check on Volume Available for Ballast

$$\begin{array}{l}\text{Volume of barge} \\ \text{available for} \\ \text{ballast water}\end{array} \cong \begin{array}{c}\text{Total volume} \\ \text{of barge}\end{array} - \begin{array}{c}\text{Volume of barge} \\ \text{used for plant}\end{array}$$

$$= (450)(600)(50) - (350)(300)(25)\ \text{ft.}^3$$

$$= 13{,}500{,}000 - 2{,}625{,}000$$

$$= 10{,}875{,}000\ \text{ft.}^3$$

$$\begin{array}{l}\text{Volume available} \\ \text{for ballast}\end{array} = \frac{(10{,}875{,}000)(64)}{2000} = 348{,}000 \text{ tons}$$

Thus adequate tank space is available for ballast required to provide a total submerged weight of 35,000 tons.

Location of Center of Gravity

$\bar{X}$ = 300'

$\bar{Y}$ = 225'

$\bar{Z}$ = 54.3' (re. barge bottom)

Center of Buoyancy (surface)

$\bar{X}$ = 300'

$\bar{Y}$ = 225'

$\bar{Z}$ = 20'

4.3.2 Transverse Stability (surfaced)

$$BM_T = I_T/\nabla^{1}$$

where

B = Center of Buoyancy

M_T = Transverse metacentric height

I_T = moment of inertia about section centerline

∇ = Volume of water displaced

$$I_T = \sum_{i=1}^{2} \frac{b_{iT} H_{iT}^{3}}{12}$$

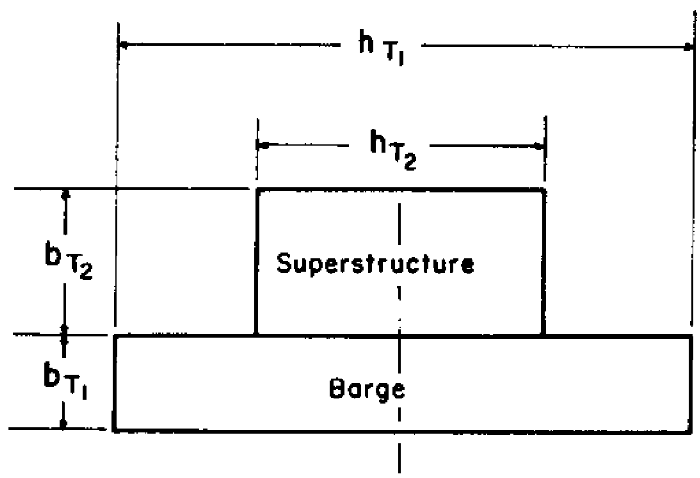
where

b_{T_1} = 50'

h_{T_1} = 450'

b_{T_2} = 175'

h_{T_2} = 300'



$$I_T = \frac{(50)(450)^3}{12} + \frac{(175)(300)^3}{12}$$

$$I_T = 7.74 \times 10^8$$

$$\nabla = (40)(450)(600) = 1.08 \times 10^7 \text{ ft.}^3$$

$$BM = \frac{(7.74)(10^8)}{(1.08)(10^7)} = 72 \text{ ft.}$$

GM_T = 92 - 54 = +38 feet (see Figure 4-7)

GZ = GM sin θ for small heeling angles (see Figure 4-8).

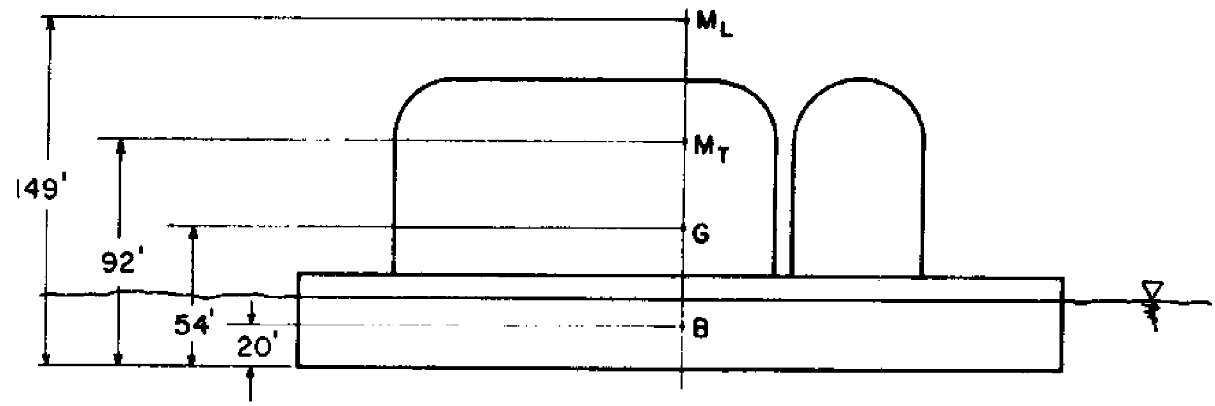


Figure 4-7. Surfaced stability parameters.

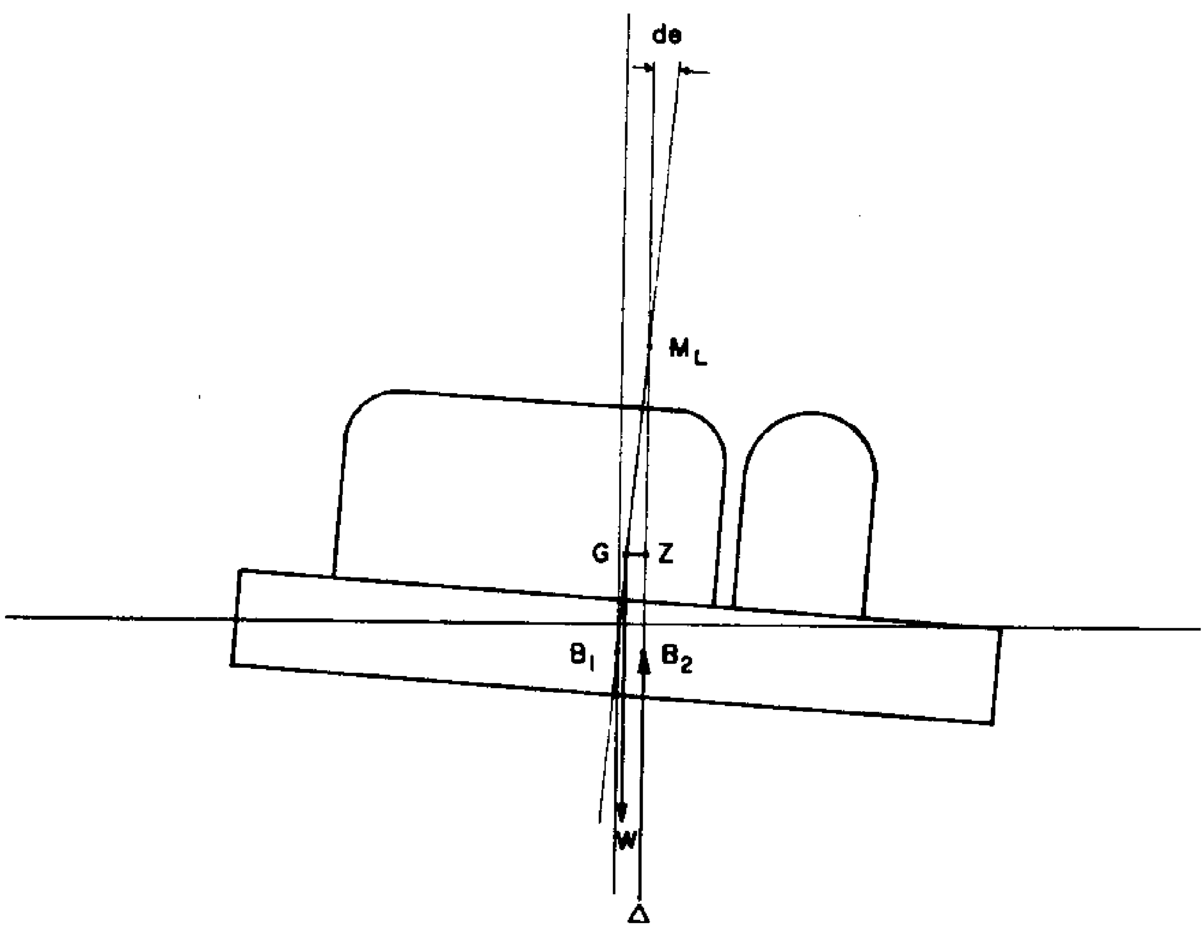


Figure 4-8. Righting moment parameters.

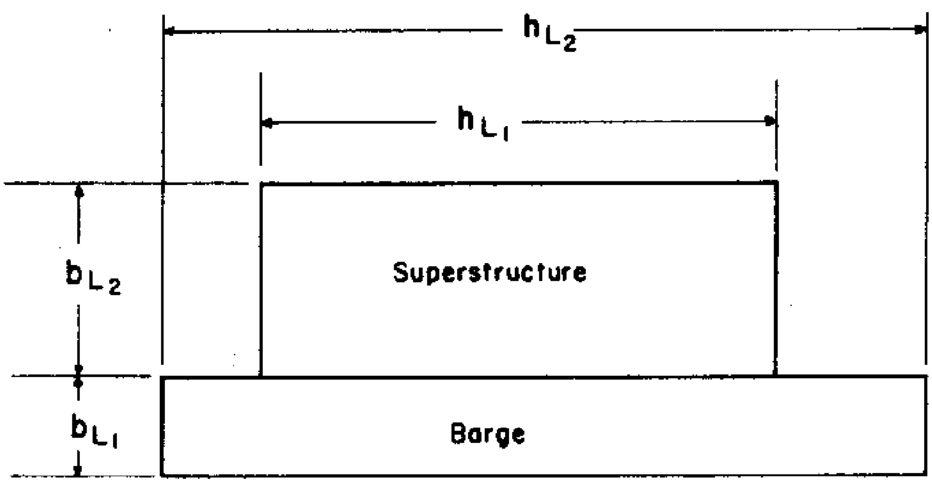
Positive stability requires GM to be a positive value, that is, M higher than G. A positive GM dictates a positive righting arm (GZ). A positive righting arm occurs when the moment of weight and buoyancy tend to rotate the ship toward an upright position.

4.3.3 Longitudinal Stability

$$BM_L = I_{L/\nabla}$$

M_L = longitudinal metracentric height

$$I_L = \sum_{i=1}^{2} \frac{b_{iL} h_{iL}^3}{12}$$



b_{L_1} = 50'

h_{L_1} = 600'

b_{L_2} = 175'

$$h_{L_2} = 350'$$

$$I_L = \frac{(50)(600)^3}{12} + \frac{(175)(350)^3}{12}$$

$$I_L = 16.1 \times 10^8$$

$$BM_L = \frac{(16.1)(10^8)}{(1.08)(10^7)} = 149 \text{ ft.}$$

$$GM_L = 149-54 = 95 \text{ ft.}$$

Large positive values for GM_T and GM_L insure that the righting moments will be large and positive surfaced stability is assured.

4.3.4 Roll Period (T_R)

$$T_R = \frac{1.108k}{(GM_T)1/2} = \frac{C_1B}{(GM_T)1/2}$$

k = radius of gyration of vessel

C_1 = 0.44 (surface ships)

C_2 = 0.37 (submarines)

B = beam width (450 ft)

$$T_R = \frac{(0.44)(450)}{(38)^{1/2}} = 32 \text{ seconds}$$

4.3.5 Stability Submerging

For the fully submerged condition; the center of buoyancy has moved upward toward the center of gravity

$$B_{\text{submerged}} = 55.5$$

$BM_T' = I_T/\nabla'$, where ∇' is now 2.3×10^7

$$BM_T' = 7.74 \times 10^8/2.37 \times 10^7 = 32.7 \text{ ft.}$$

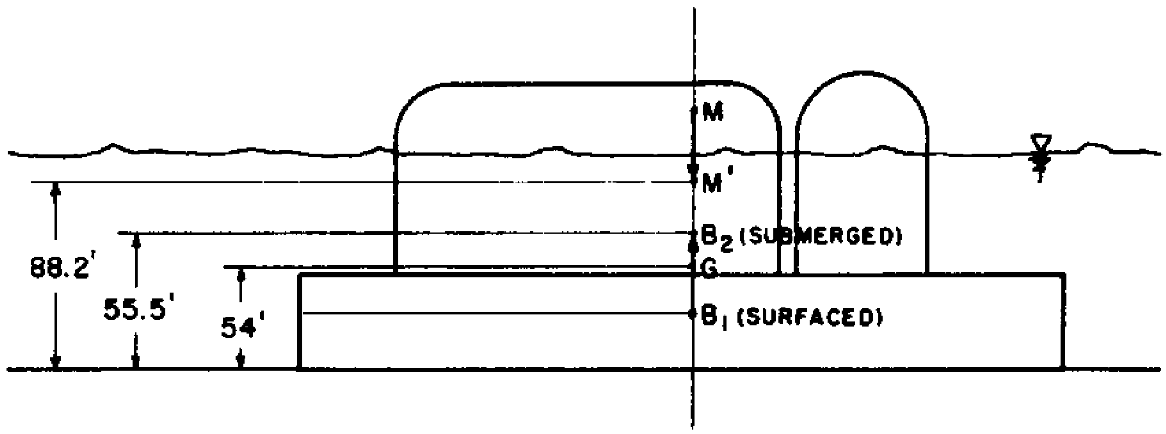


Figure 4-9. Submerged stability parameters.

During submergence, the center of buoyancy rises, and the metacenter becomes lower. However, the metacentric height GM remains positive (see Figure 4-9).

$$GM_{min} = GB + BM_T = 1.5' + 32.7 = 33.9'$$

Hence, a positive righting arm will be developed for tipping at any stage during submergence.

4.4 CONSTRUCTION, LAUNCHING AND TRANSPORTATION TO SITE

In this section the problems and alternatives of construction, launching and transportation of a large barge-like structure are considered.

4.4.1 Construction

As discussed in previous chapters, the next 30 years will undoubtedly be characterized by increasing dependence on nuclear plants for power. The increased concern for public safety will therefore dictate that plant sites be as remote as possible. It is therefore not unreasonable to assume then that the number of offshore sitings may become large.

The requirements of a large-sized support barge, handling of heavy power plant components, and siting at sea narrow down the choices of a construction site to either an existing coastal shipyard, or a newly constructed onshore facility close to the desired site. It is felt that for the latter, costs of construction would be orders of magnitude higher, and the launching operation (extremely long and massive ground ways and/or extensive towing power to pull the barge to waters deeper than the 40 foot draft) might not be feasible and if so would be very costly.

There are, however, many major shipyards in the United States equipped with heavy duty cranes that are capable of transferring the power plant components to the barge.

Construction of multiple offshore plants in a single coastal shipyard would provide many economic advantages. The plant design could remain essentially the same with modifications being primarily in the type and extent of the foundations constructed at each site.

4.4.2 Launching

Standard launching procedures include end launching (typical at coastal shipyards), side launching (rivers and Great Lakes), and undocking from a flooded graving dock. The graving dock approach presents the least expensive and most efficient approach to the launching of the completed power plant facility--provided the dock exists. Additional advantages to graving dock construction include ease in equipment transfer to and from the barge, simplicity in construction of an even keel, and high accessibility to the vessel even when construction is nearly completed.

Launching from a graving dock involves flooding of the dock itself, releasing mooring cables, and towing the completed structure away from the dock. Even with only a single large graving dock, it would be possible to have more than a single plant under construction at any one time. Upon completion of the barge, it could be launched and the remainder of construction completed dockside. This would free the graving dock for commencement of a second barge.

4.4.3 Transportation to Site

Alternate modes of transportation to the site include towing by one or more ships, pushing the barge as is typically done on many rivers, or providing an internal propulsion mechanism. Towing and pushing of large barges in the open ocean has proven hazardous and only marginally successful. Providing an internal propulsion system for a single trip would in all probability not be economically practical. On the other hand, if multiple power plants of similar configuration are to be constructed it might be worthwhile to make the barge seaworthy by the addition of a temporary faired bow and stern propulsion unit (Figure 4-10). Such an arrangement would provide the increased maneuverability of a large ship. Upon arrival at the proposed site, the increased positioning capability will aid in properly aligning the barge prior to flooding the cells. Once the barge is properly positioned and the foundation cables attached, the false bow and propulsion units are separated from the barge and joined to form a single vessel. This ship can then return to the shipyard for transportation of another power plant to its designated site.

4.5 COOLING SYSTEM DESIGN

The overall factors considered in a plant heat/energy budget are depicted in Figure 4-11.

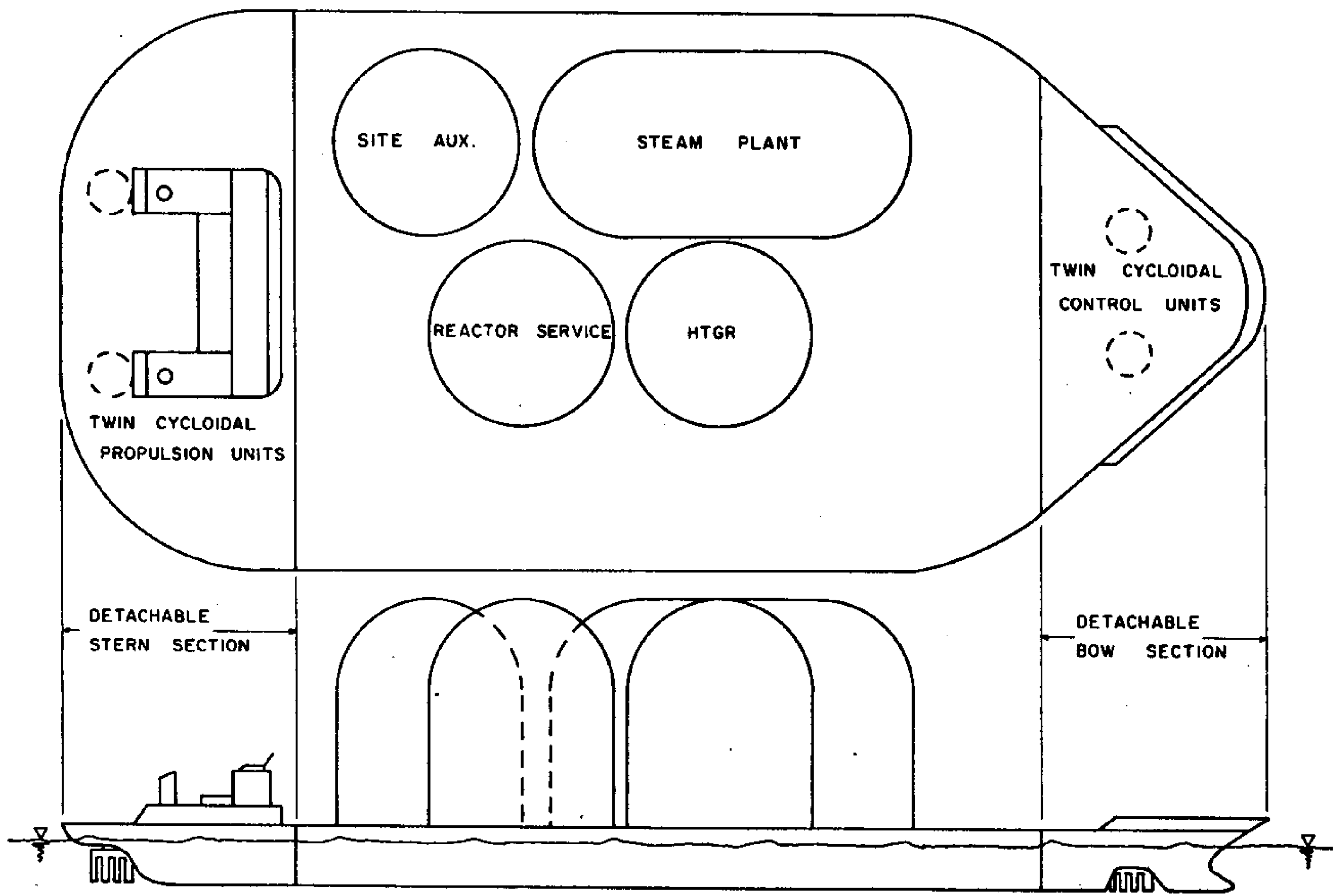


Figure 4-10. Configuration for transportation to site.

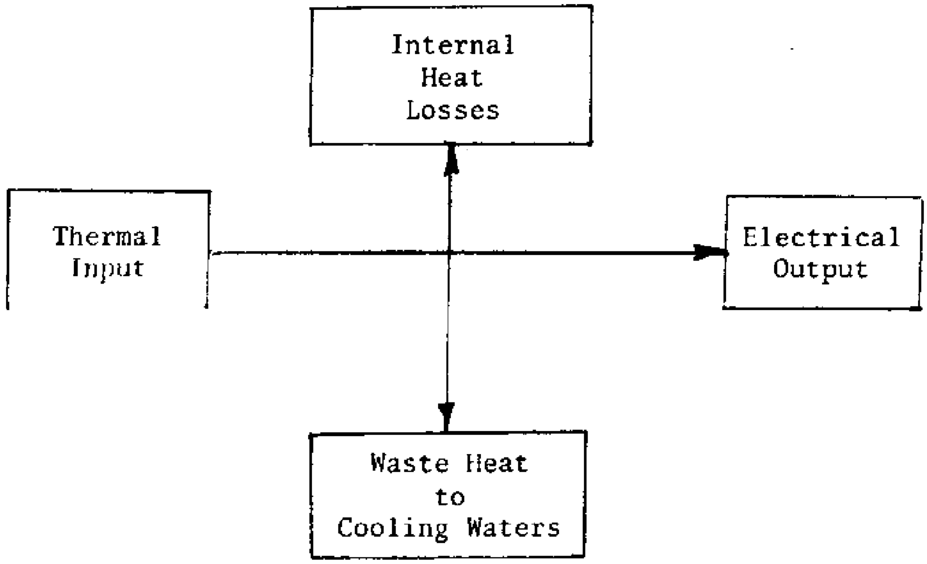


Figure 4-11. Plant heat/energy budget.

In equation form, these items are related by:

$$H_{IN} = E_o + H_W + H_{IL} \quad \text{(BTU/HR.)} \qquad (1)$$

where

H_{IN} = Thermal imput rate

E_o = Electrical output rate

H_W = Rate of waste heat discharged to cooling waters

H_{IL} = Internal heat loss rate

The overall plant thermal efficiency is defined by [38]

$$\eta_t = \frac{E_o}{H_{IN}} \quad \text{(dimensionless)} \qquad (2)$$

For nuclear power plants employing high temperature gas reactors, η_t = 39% and internal heat loss is approximately 5% of the net thermal input.[6] Solving for the rate of injection of waste heat into the cooling water in equation (1) gives:

$$H_W = H_{IN} - H_{IL} - E_o \qquad (3)$$

$$H_W = H_{IN} - 0.05\, H_{IN} - E_o \qquad (4)$$

$$H_W = 0.95\ H_{IN} - E_o \ldots \quad (5)$$

$$H_W = 0.95\ (E_o/.39) - E_o \ldots \quad (6)$$

The energy equivalent to one KWH is 3413 BTU. Thus:

$$H_W = 0.95\ (3413/.39) - 3413 \ldots \quad (7)$$

$$H_W = 4692\ \text{BTU/KWH} \ldots \quad (8)$$

For an 1100 megawatt HTGR nuclear power generation facility, the heat energy discharge rate to the cooling waters will be

$$H_W = (4692\ \frac{\text{BTU}}{\text{KWH}})\ (1.1 \times 10^6\ \text{KW}) = 5.16 \times 10^9\ \frac{\text{BTU}}{\text{HR}} \ldots (9)$$

Flow Rate vs. Resultant Temperature Rise

$$H_W = (5.16)(10^9)\ \frac{\text{BTU}}{\text{HR}} \times \frac{\text{LB-°F}}{\text{BTU}} \times \frac{\text{FT}^3}{64.0\text{LB}} \times \frac{5°\text{C}}{9°\text{F}}$$

$$\times \frac{H_R}{3600\ \text{SEC}} \ldots \quad (10)$$

$$H_W = 1.245 \times 10^4\ \frac{\text{FT.}^3 - °\text{C}}{\text{SEC.}} \ldots \quad (11)$$

This equation is plotted in Figure 4-12 to facilitate choice of an appropriate flow rate for any desired temperature rise in the once-through cooling water system.

4.5.1 Site Sea Water Temperature Data

Ten year mean monthly water temperatures at a hydrographic station located three nautical miles off the Oregon coastline are given in Table 4-I.

These data indicate that the maximum temperature difference between surface and waters at 100 feet deep will be approximately 4°C. Throughout most of the year the temperature differential is less than 2°C and approaches a minimum value during the months of February and March.

Threshold temperatures at which biological problems seem to change in magnitude are 18.3°C and 32.2°C.[39] The surface water temperatures at the site vary from approximately 8 to 12°C.[39] This would indicate that the controlling criteria will be a reasonable ΔT above ambient rather than a maximum T of the water at any one point.

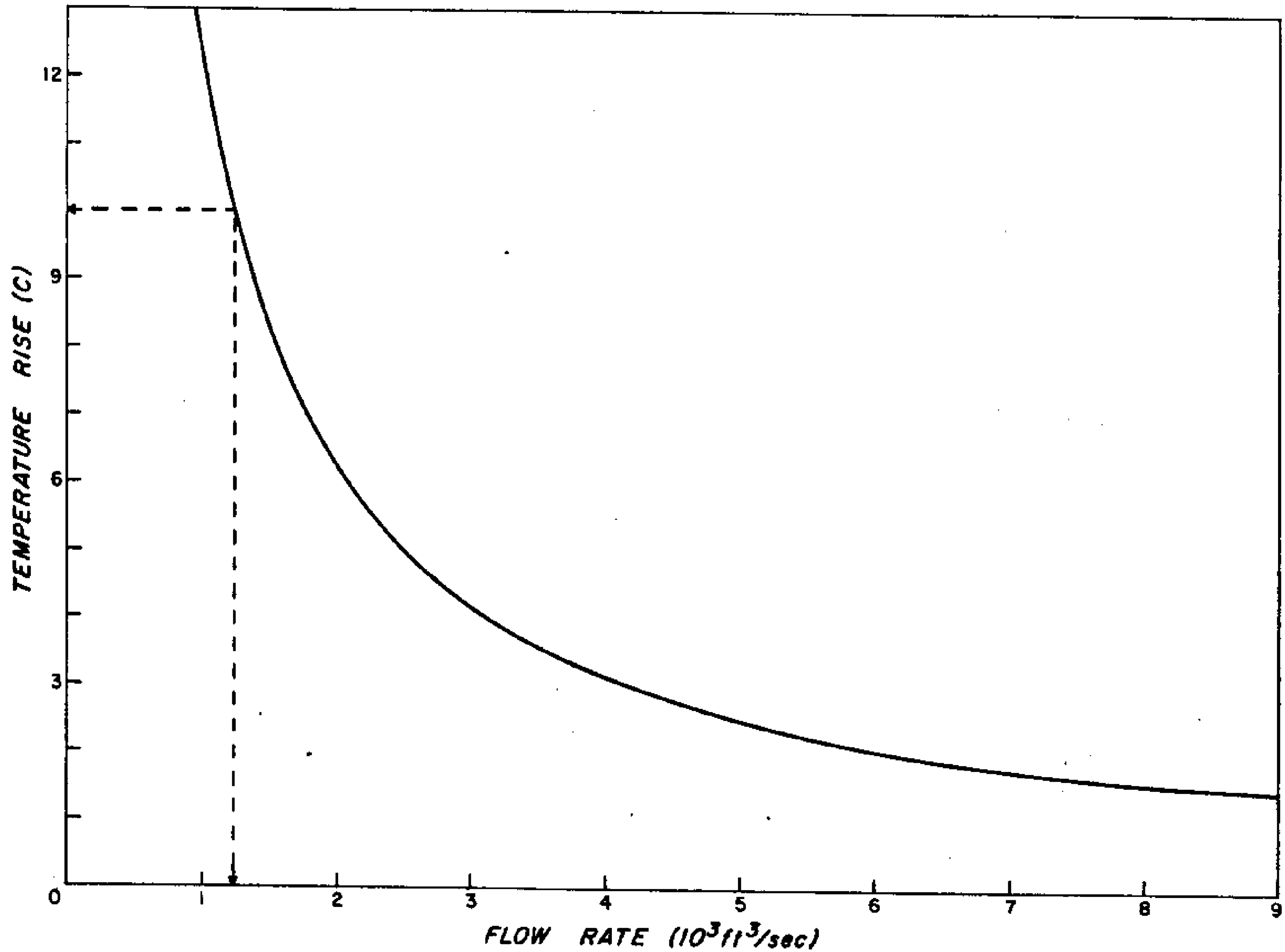


Figure 4-12. Discharge rate (R) as a function of temperature rise (ΔT).

Month	Surface	30 Meters	$\Delta T(^{\circ}C)$
Jan.	8.44	9.59	1.15
Feb.	8.79	8.64	0.15
Mar.	9.10	9.28	0.18
Apr.	10.95	9.15	1.80
May	9.32	7.59	1.73
June	10.65	6.70	3.95
July	8.57	7.43	1.14
Aug.	9.67	7.30	7.39
Sept.	11.67	8.13	3.54
Oct.	12.01	10.96	1.05
Nov.	11.56	11.49	0.07
Dec.	10.89	11.01	0.12

Table 4-I Mean monthly sea water temperatures 3 nautical miles off the Oregon coast..

4.5.2 Cooling System Design Criteria

To insure a minimum impact due to the cooling water discharge, the design criteria chosen for this project are:

1. ΔT_{max} = $2^{\circ}C$ at the plume center when it reaches the ocean's surface.

2. ΔT_{max} above ambient = $0.5^{\circ}C$ at a radius of 500 feet from the central point of the surfaced plume.

3. The smaller of these two criteria will be at any time considered the legal requirement.

The temperature difference (ΔT) between the intake and discharge cooling waters is a function of plant size, thermal efficiency, condenser type and cooling water pumping capacities. A once through cooling system will be considered for this study. For such a system the surface area, and hence size, of the condenser is inversely proportional to the log mean of the temperature difference across it.[40] The operating back pressure on the turbine, which is directly related to plant operating efficiency, is also a function of the condenser ΔT. The larger this value, the lower the turbine back pressure and the higher the conversion efficiency. However, the temperature-efficiency relationship is not linear and below certain temperatures the efficiency gain is more than offset by the additional costs for larger capacity pumps and piping. A summary of flow rates, ΔTs and thermal discharge temperatures for nuclear plants employing once through cooling is given in Table 4-II. Reference will be made to this table to insure that the design flow rates are reasonable compared to those employed by other nuclear plants of similar size.

Because of the small vertical temperature gradients normally encountered, intakes may be located deep to maintain the minimum possible inlet temperature year round. The deeper the location of the discharge pipe, the longer will be the period of travel to the surface and the greater the turbulent mixing and dilution. Hence, location of the discharge a minimum distance above the intakes is the optimal spacing. The possibility of feedback of warmed water into the intake under inversion conditions, however, must be avoided. This should also be considered in determining the vertical separation of the two pipes.

4.5.3 Thermal Plume Prediction

One approach to a cooling system design is to begin by specifying the allowable extent of the mixing zone and the temperature distributions both inside and outside this zone. One could then use prediction models and theories in a reverse fashion to determine the outlet configuration and orientation, and the maximum acceptable temperature at this point. This information would then dictate (based on a known amount of energy to be discharged) what the water velocities, pipe diameters and condenser type and size need be.

For purposes of this conceptual study, a mixing zone of 500 foot radius will be specified. At the outer limit of this zone, the maximum temperature rise above the average daily surface temperature will be less than 0.5°C.

*Data from "Thermal Effects Studies by Nuclear Plant Licensees & Applicants -- 1969 (U. S. AEC.)

Plant	Mw(Thermal)	Mw(Elec.)	Reactor Type	Condenser ΔT(°C)	Cooling Water Flow Rate cfs	Discharge Water Temperature Data/Controlling Regs.
Arkansas Nuc. 1	2452	850	PWR	8.3	1700	Requirements outside mixing zone ΔT increase any stream = <5°F. <68° F - Trout Streams; <86° F - Small Mouth Bass Streams; <95° F - All others
Calvert Cliffs 1 Calvert Cliffs 2 (Maryland)	2450 2452	800 800	PWR PWR	5.5	5400 Both	Temp. elevation < 10° F above natural water temperature Temp. max. = 90° F outside state designated mixing zone
Pilgram Stn. (Massachusetts)	1912	625	BWR	15.5	695	Will meet state criteria approved by Secy. Interior
Brunswick 1 Brunswick 2 (North Carolina)	2436 2436	821 821	BWR BWR	10.5	3000 Both	State water criteria under review by Secy. Int. (1969)
H. B. Robinson 2 (South Carolina)	2200	700	PWR	10.0	1070	Discharge temp. (most adverse conditions) will be 115° F from the discharge canal into the lake
Dresden 1 Dresden 2 Dresden 3 (Illinois)	700 2527 2527	200 809 809	BWR BWR BWR	?	2660 total	Units 2 & 3 will employ a cooling lake to meet state temperature criteria; Illinois R. will not exceed 90° F after mixing nor be increased > 5° F above ambient
Zion 1 Zion 2 (Illinois)	3391 3391	1085 1085	PWR PWR	10.9	3260	Applicable state water criteria designate T. Max. 85° F
Quad Cities 1 Quad Cities 2 (Illinois)	2511 2511	809 809	BWR BWR	?	2230	Mississippi R. temp. not to exceed 90° F after mixing nor be increased > 5° F above ambient
Conn. Yankee (Connecticut)	1825	567	PWR	10.5 - 12.2	830	Max. temp. 85° F & Max. rise of 4° F <u>in</u> mixing zone
Indian Point 1 Indian Point 2 Indian Point 3 (New York)	615 2758 3025	265 873 965	PWR PWR PWR	7.7	4550	No increase > 90° F surface T. at any point. 50% vol. flow of estuary (including 1/3 of surface water) to be < 4° F rise or T. max. 85° F - whichever is less. July to Sept. if surface T > 83° F, max. rise permitted = 1.5° F.
Nuc. Unit 4 Nuc. Unit 5 (New York)	3293 3293	1115 1115	BWR BWR	9.3	3814	

Table 4-II TEMPERATURE DATA -- NUCLEAR POWER PLANTS WITH SINGLE PASS COOLING

Plant	Mw(Thermal)	Mw(Elec.)	Reactor Type	Condenser ΔT(°C)	Cooling Water Flow Rate cfs	Discharge Water Temperature Data/Controlling Regs.
Bell Station (New York)	2436	838	BWR	11.1	1100	Apr. 1964 Co. announced postponement for cooling systems and thermal discharge research
Nine Mile Pt. (New York)	1850	620	BWR	17.7	558	N.Y. permit allows 250,000 GPM cooling water with max. rise 32° F (Lake Ontario)
Ft. Calhoun 1 (Nebraska)	1420	457	PWR	11.4 (10 to river)	802 (702 condensers) (100 aux. coolers)	Max. T. rise 5° F May to Oct. Max. T rise 10° F Nov to Apr. (Missouri R.) Max. T. absolute 90° F Max. T. rate 2° F/Hr Mixing zone size to be determined after construction
Humbolt Bay 3 (California)	240	68.5	BWR	8.3	115	
Diablo Canyon 1 Diablo Canyon 2 (California)	3250 3250	1060 1060	PWR PWR	10	1930	
Peach Bottom 1 Peach Bottom 2 Peach Bottom 3	115 3294 3294	40 1065 1065	HTGR BWR BWR	 11.5 11.5	100 3350 TOT 243	10° F ΔT at discharge into Susquehanna R. due to cooling tower assist
Fitzpatrick (New York)	2436	850	BWR	32.4	825	Discharge into L. Erie at < 3° F above ambient. Surf. T rise < 3° F beyond radius of 300' or equiv. area.
Salem 1 Salem 2 (New Jersey)	3250 3250	1050 1050	PWR PWR	7.5	4850	Max. T. 86° F & max. increase 5° F above Avg. daily R. temp.
San Onofre (California)	1347	430	PWR	10	780	Largest measured T. increase at surface boil = 9° F; Generally < 6° F; Surface area heated to 4° F. above nat'l. temp. confined to a one to five acres.
Browns Ferry 1 Browns Ferry 2 Browns Ferry 3 (Tennessee)	3293 3293 3293	1098 1098 1098	BWR BWR BWR	14	4400 Tot	Designed to limit mixing zone to < 75% total cross section of wheeler reservoir; outside mixing zone, limit Avg. T < 93° F and at any point in the cross section to < 95° F; limit T. rise to < 10° F above nat'l. T.
Point Beach 1 Point Beach 2 (Wisconsin)	1518 1518	497 497	PWR PWR	10.7 -17.5	712 - 1560	High ΔT & low flow when lake T < 40° F Low ΔT & high flow when lake T > 40° F

Table 4-II TEMPERATURE DATA -- NUCLEAR POWER PLANTS WITH SINGLE PASS COOLING (Cont.)

Plant	Mw(Thermal)	Mw(Elec.)	Reactor Type	Condenser ΔT(°C)	Cooling Water Flow Rate cfs	Discharge Water Temperature Data/Controlling Regs.
Palisades (Michigan)	2212	700	PWR	15.5	892	
Enrico Fermi Unit 2 (Michigan)	3293	1126	BWR	10	2000	Water will be raised 18° F over cond; 2° F lost to lagoon and water returned to Lake Erie will be 16° F above ambient
Crystal R. 3 (Florida)	2452	825	PWR	9.7	1560	
Turkey Pt. 3 Turkey Pt. 4 (Florida)	2097 2097	652 652	PWR PWR	6.6	1490	6 mile canal to card sound
Hutchinson Is. Unit 1 (Florida)	2440	825	PWR	7.7 - 10.0	1670	
D. C. Cook 1 D. C. Cook 2 (Michigan)	3250 3250	1054 1060	PWR PWR	11.6	3500	
Oyster Cr. 1 (New Jersey)	1600	530	BWR	9.1	1040	Will decrease ΔT discharge to 10° F via dilution pumps in summer
Shoreham Stn. (New York)	2436	819	BWR	10.8	1340	
Malibu Unit (California)	1473	462	PWR	11.1	790	Discharge 1000' offshore; max. ΔT 1/2 mile from discharge 1° F above ambient.
Maine Yankee (Maryland)	2440	790	PWR	14.2	950	
Millstone Pt. 1 Millstone Pt. 2 (Connecticut)	2011 2560	652 828	BWR PWR		935	Plants will meet state reqmts. of max. T 85° F; max. rise 4° F over natural water temperature
Cooper Nuc. Stn. (Nebraska)	2381	778	BWR	10	1450	

Table 4-II TEMPERATURE DATA -- NUCLEAR POWER PLANTS WITH SINGLE PASS COOLING (Cont.)

Many authors have treated the problem of describing the distribution and dilution characteristics of buoyant plumes (references 41, 42, 43, 44, 45). Liseth[42] presents a summary of such efforts and indicates that relatively good agreement between these authors exists. The central temperature of the heated discharge plume at the waters surface can be estimated as follows:

$$S_m = \frac{t_o - t_r}{t_m - t_r} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

where

S_m = dilution along the jet axis

t_o = temperature of the discharge water

t_r = temperature of the receiving water

t_m = temperature along the centerline of the jet

also

$$F_o = \frac{U_o}{[(\frac{\rho_r - \rho_o}{\rho_o})\ g\ D_o]^{1/2}} = \frac{U_o}{[g'D_o]^{1/2}} \quad \ldots\ldots \quad (13)$$

where

F_o = densimetric Froude number

U_o = discharge velocity of the jet

ρ_r = density of the receiving water

ρ_o = density of the discharge water

g = acceleration due to gravity

g' = acceleration due to gravity corrected for buoyancy

D_o = diameter of jet at the outlet

Liseth[42] presents a graph (Figure 4-13) of Y/D_o versus F_o from which one can determine S_m (Y represents the vertical distance of interest above the point of discharge). Having calculated S_m, and the values of ρ_o and ρ_r the temperature at the plume center t_m can be calculated for any Y.

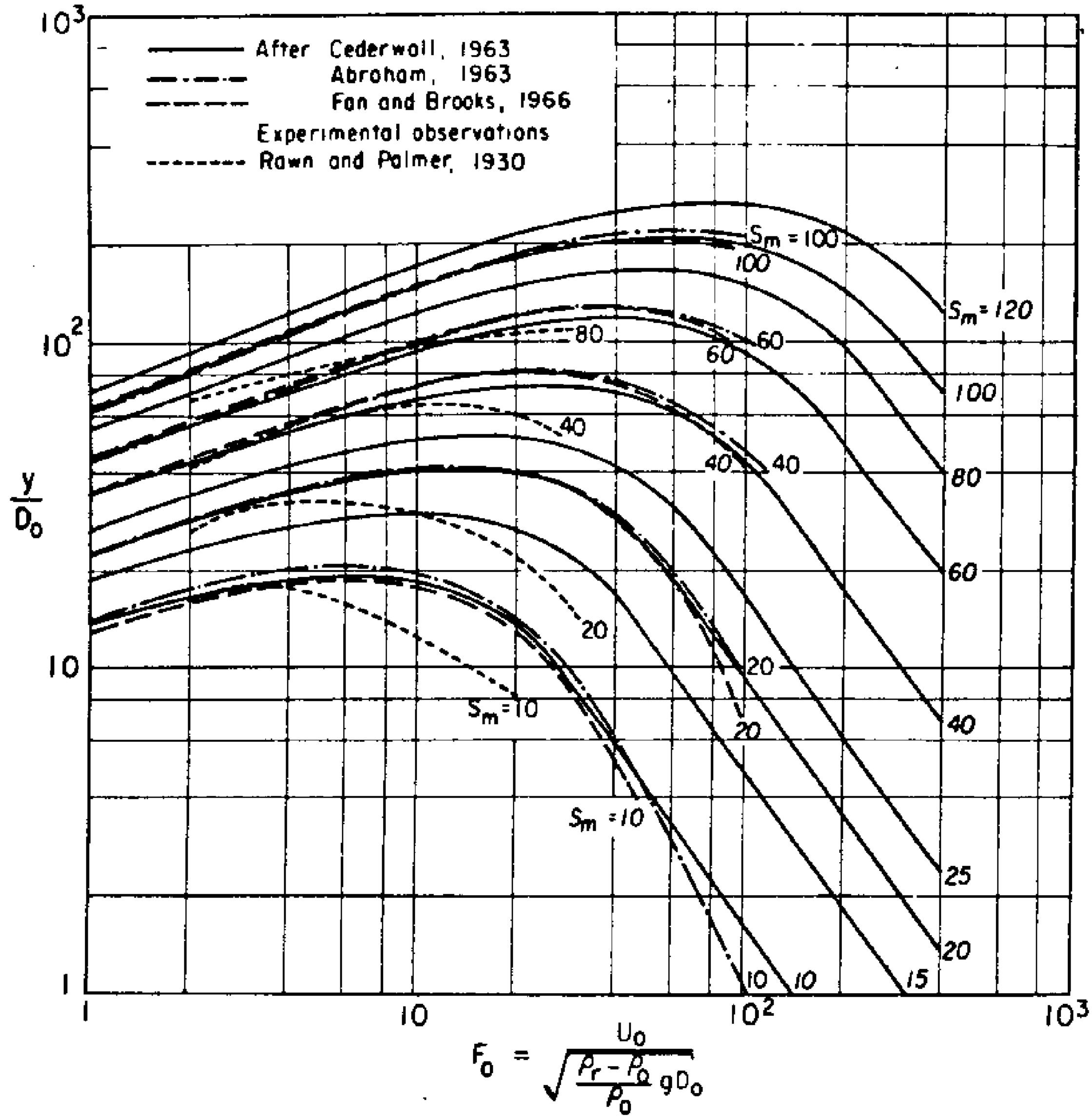


Figure 4-13. Theoretical solutions of dilution S_m for horizontal buoyant jet in stagnant receiving water of uniform density (from Liseth, 42)..............................

For example, assume:

Y = 100 ft. distance below surface of discharge pipe

D_o = 10 ft. discharge pipe diameter

ΔT = 10°C across the condenser

then:

$Y/D_o = 10$

U_o = discharge rate (R)/pipe cross sectional area (A)

R (from Figure 4-12) = 1250 ft.^{3}/sec.

$$U_o = \frac{1250 \text{ ft.}^3/\text{sec.}}{(3.14)(25)\text{ ft.}^2} = 15.9 \text{ ft./sec.}$$

ρ_r = 1.99284 (assuming t_r = 9°C)

ρ_o = 1.98886 (assuming t_r = 9°C)

$$F_{o_1} = 15.9/ \quad \frac{(0.00398)}{1.98886} \quad (32.2)(10) \quad ^{1/2} \quad = 19.75$$

Employing Figure 4-13 for Y/D_o = 10 and F_o = 19.75, S_m = 10. Hence from equation (12):

$$t_m = (t_o - t_r)/(10) + t_r$$

$$t_m = (19-9)/(10) + 9$$

$$t_m = 10°\text{C}$$

Thus, the temperature at the plume center at the water surface will be 1°C above ambient which is within the established designation of a maximum ΔT of 2°C.

The flow rate is high, however, and to establish a reasonable flow, two 10 foot diameter pipes are considered. Such a configuration will offer redundance in event of a partial breakdown or requirement for repair on part of the system.

Consider:

$$Y = 107'$$

$$\text{each } D_o = 10$$

$$T = 10°C$$

$$Y/D_o = 10.7$$

$$R = 625 \text{ ft.}^3/\text{sec.}$$

$$U_o'(\text{discharge each pipe}) = U_o/2 = 7.95 \text{ FPS}$$

$$F_{o_2} = F_{o_1}/2 = 9.8$$

hence from Figure 4-13

$$S_m = 7.0$$

$$t_m = \frac{(19-9)}{5} + 9 = 11°C$$

Hence the upper limit of the design criteria has been met. To insure that t_m is less than 2 degrees above ambient requires a lower ΔT across the condensers and therefore a higher cooling water flow rate. Assuming a $\Delta T = 9°C$, the flow rate required is 1400 cfs. Repeating the above calculation indicates that for two 10' diameter discharge lines and a flow velocity of 10.45 fps in each, the surface temperature at the center of each plume will be 10.4°C.

As a verification of the previous calculation and the determination of the change in plume diameter and lateral displacement by the time it reaches the surface -- the procedure of Fan and Brooks,[41] is used.

$$m_o = \left\{\frac{g\lambda^2 U_o{}^3 b_o{}^6 (\rho_o - \rho_1)}{4\sqrt{2}\,\alpha\rho_o}\right\}^{-2/5} \quad \frac{U_o{}^2 b_o{}^2}{2}$$

where

$$m_o = \left\{\frac{g\lambda^2 U_o{}^3 b_o{}^6 (\rho_o - \rho_1)}{4\sqrt{2}\,\alpha\rho_o}\right\}^{-2/5} \quad \frac{U_o{}^2 b_o{}^2}{2}$$

$$\alpha = \text{entrainment coefficient} = 0.082$$

$$\lambda = (\text{Schmidt number})^{1/2} = 1.16$$

b_o = $D/\ 2$ = characteristic length defined by velocity profile

U_o = velocity at jet discharge

ρ_o = reference water density

ρ_1 = discharged water density at discharge

D = pipe diameter

Again assume

V_o = 8.4 ft./sec.

ρ_o = 1.99284

ρ_1 = 1.98886

$\rho_o - \rho_1 = 0.0039°C = \Delta\rho$

b_o = 10/11414 = 7.07

Then:

$$m_o = \left[\frac{(32.2)(1.34)(5.92)(10^2)(1.28)(10^5)(3.98)(10^{-3})}{(4)(1.414)(8.2)(10^{-12})(1.99)}\right]^{-2/5}$$

$$\frac{U_o^2 b_o^2}{2}$$

$$m_o = [1.41 \times 10^7]^{-2/5} \quad \frac{(7.06)(10^1)(5)(10^1)}{2}$$

$$m_o = (1.38)(10^{-3}) \quad (1.76)(10^3)$$

$$m_o = 2.42$$

$$\xi m_o = 2\sqrt{2}\ \alpha Y/D$$

$$\xi m_o = 2\ (1.414)(8.2)(10^{-2})(1.07)(10^1)$$

$$\xi m_o = 2.48$$

Using Figure 4-14:

$$\delta_o = 7.2$$

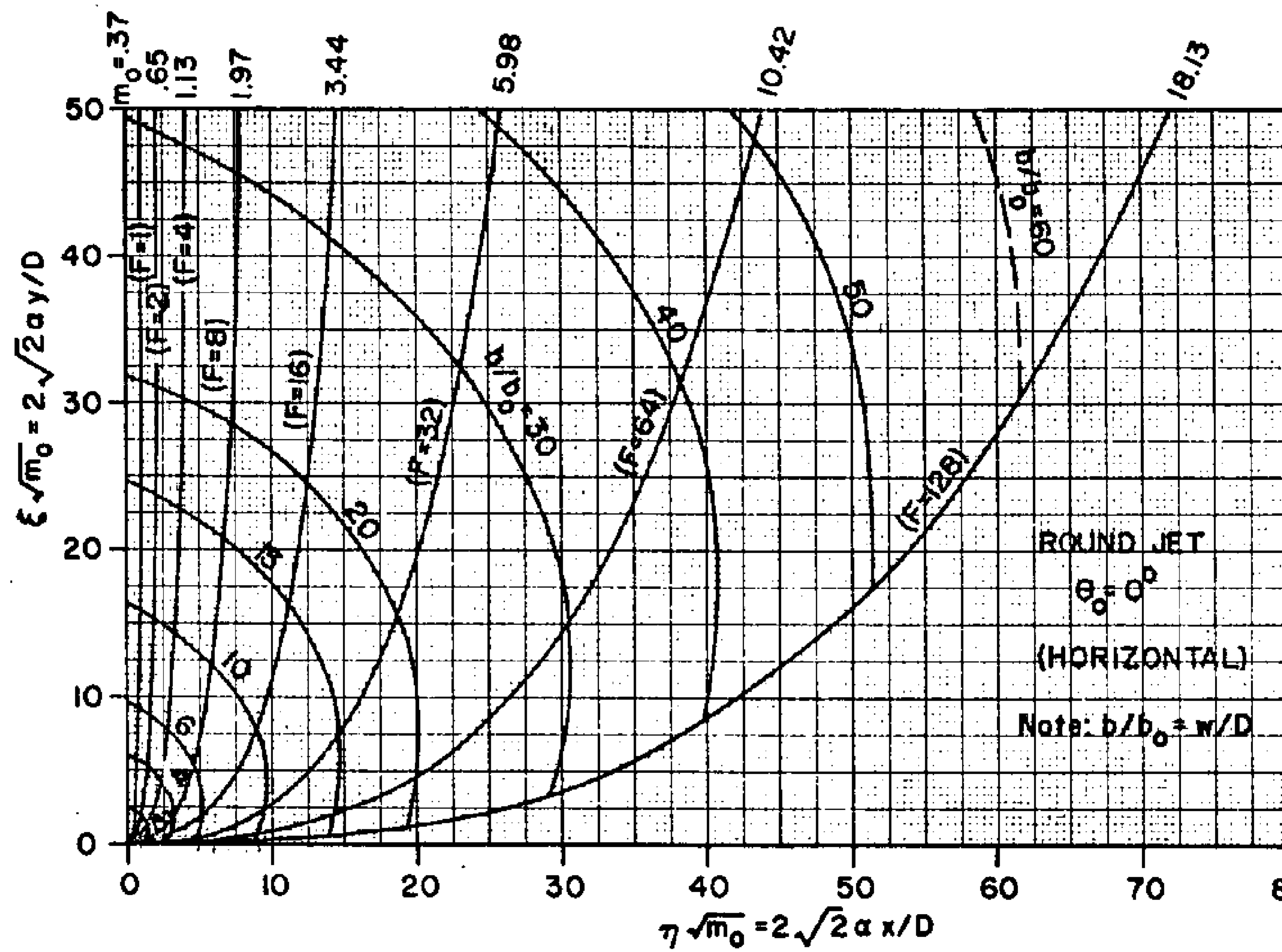


Figure 4-14. Trajectories and half-width b/b_o of round buoyant jets in stagnant uniform ambient fluids: $\theta_o = 0^o$ (Froude numbers (F) shown are based on $\alpha = 0.082$) (from Fan and Brooks, 41).

and $t_m = \frac{(19-9)}{7} + 9 = 10.4°C$

Using Figure 4-15:

$$\frac{2\sqrt{2}\,\alpha x}{D} = \frac{(4)\ (10)}{(2)(1.414)(8.2)(10^{-2})} = 1.725 \times 10^2$$

$$x = 172 \text{ ft.}$$

$$\frac{b}{b_o} = 3$$

$$b = 3b_o \text{ or } \frac{D'}{1.414} = \frac{3(10)}{1.4.4} \qquad D' = 30'$$

These calculations indicate that the surface temperature at the plume center will be 10.4°C or 1.4°C above the assumed ambient of 9°C. This is well within the design criteria of 2.0°C above ambient. During the summer months the intake waters will be less than 9°C (i.e. in June 6.7°C due to upwelling) and the discharge temperature will be 15.7°C. For a dilution ratio of 7.5, and assuming a worst case where the warmer surface temperature of 10.65°C persists to the depth of discharge,

$$t_m = (t_o - t_r)/S_m + t_r$$

$$t_m = 15.7 - 10.65/7.5 + 10.65$$

$$t_m = 11.32°C$$

or

Δt = 11.32 - 10.65 = 0.67°C above ambient, which remains within the design criterion.

During January, the deeper waters are warmer (9.59°C) relative to the surface (8.44°C). For this case, the discharged water will be 18.59°C and again assuming a worst case where the warmer temperatures persist to the surface,

$$t_m = 18.59 - 9.59/7.5 + 9.59 = 10.79°C$$

Thus, the plume center temperature could at worst be 10.79 - 8.44 = 2.35°C above ambient. However, the temperature gradient of 1.15°C in going to the surface will more than offset the 0.35°C above design criterion.

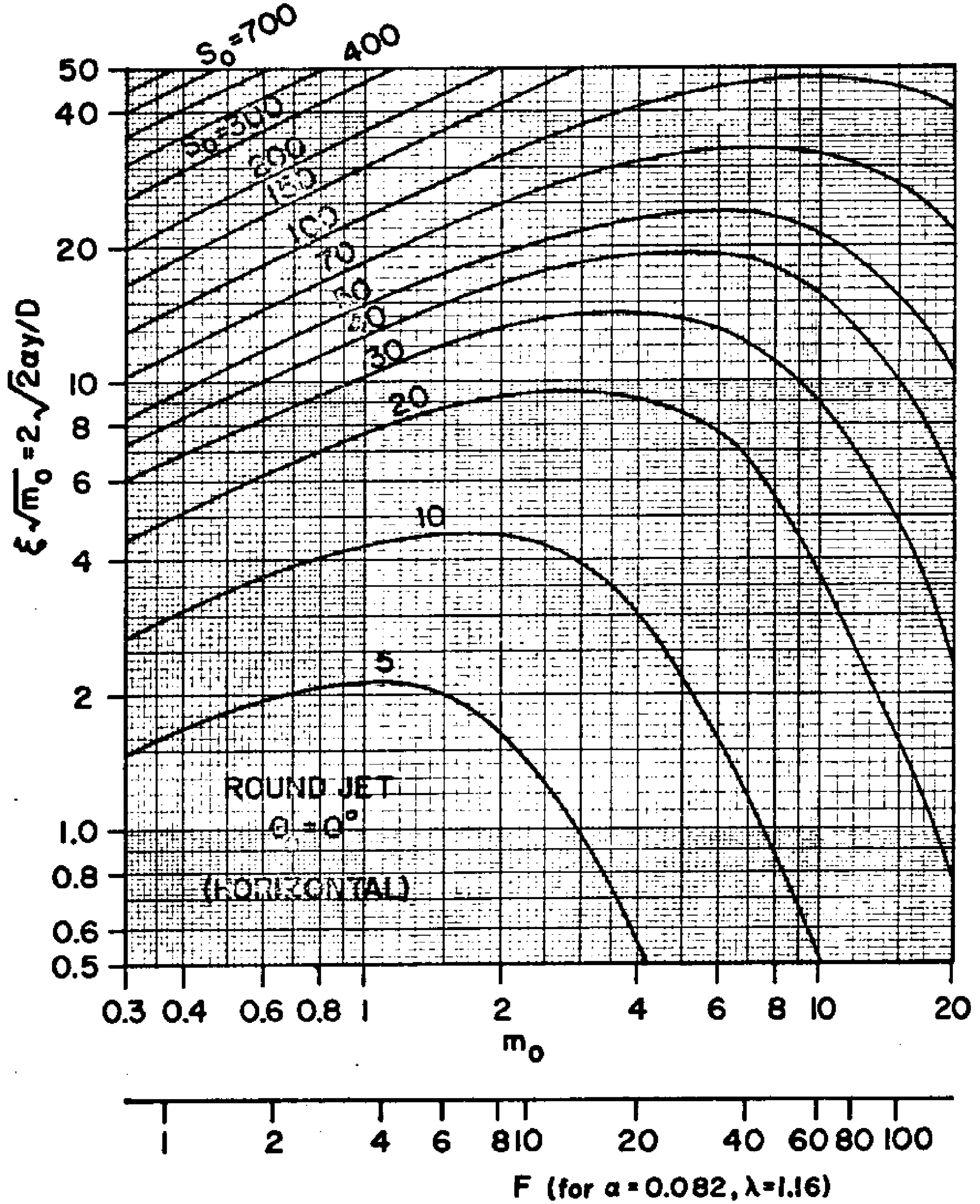


Figure 4-15. Dilution of round buoyant jets in stagnant uniform environments: $\theta_0 = 0^o$ (from Fan and Brooks, 41).

The surface diameter of the plume is approximately 30 feet. As shown in Figure 4-16, the lateral separation between the two 10 foot diameter discharge pipes is 120 feet. This will insure no overlap in the turbulent mixing of the rising plumes. Once at the surface, lateral diffusive and advective spreading will be a function of sea state, winds, and surface currents. A detailed analysis of the spreading rate is beyond the scope of this report. However, on the basis of temperature surveys (towing a thermister) of power generation facilities along the Southern California coastline, it is felt that the 500 foot mixing zone boundary temperature of 0.5°C above ambient is achievable for this design. In the field surveys, ΔTs of up to 10°C were measured, and often the heated surface waters were no longer detectable (to 0.1°C) beyond distances of 1000 feet from the plume center.

The intake orifices are located on the seaward side of the plant, while the thermal discharge outlets are on the east or coastal side of the plant. The 600 foot lateral separation insures non interference or recycling of heated waters. The predominant winds and currents are alongshore and hence discharge towards shore will most often cause the surface dispersion to progress away from the plant. Maintenance of heated waters near the plant would enhance biological growth and hence excessive fouling on the external structures.

The high velocity flows are decreased at the pipe entrances and exits by increasing the O.D. to 30 feet via a tapered section at all locations. Cupra-nickel gratings are provided at all pipe openings to keep sea life out of the cooling system waters. Periodic reversal of the elevated temperature flows will be required to reduce biological fouling within the intake pipes. (See sections 4.2.1 and 4.7)

4.6 MARINE CORROSION

The components of the power plant system subject to marine corrosion include materials in the air and surface zones, submerged materials and those embedded below the sea floor. Each of these zones typically has unique problems associated with it.

The primary exterior surface material used in the design is concrete, which has well known corrosion resistance characteristics. Avoiding exposed metal components eliminates to a large extent the requirements to employ stainless steels or other costly copper-nickel alloys which are highly corrosion resistant.

Particular types of corrosion which occur on structures in the marine environment include pitting, attacks in crevices (due to the formation of metal-ion or oxygen concentration cells), attacks under marine deposits, intergranular corrosion, impingment attack, stress corrosion cracking, corrosion fatigue, galvanic effects, dezincification

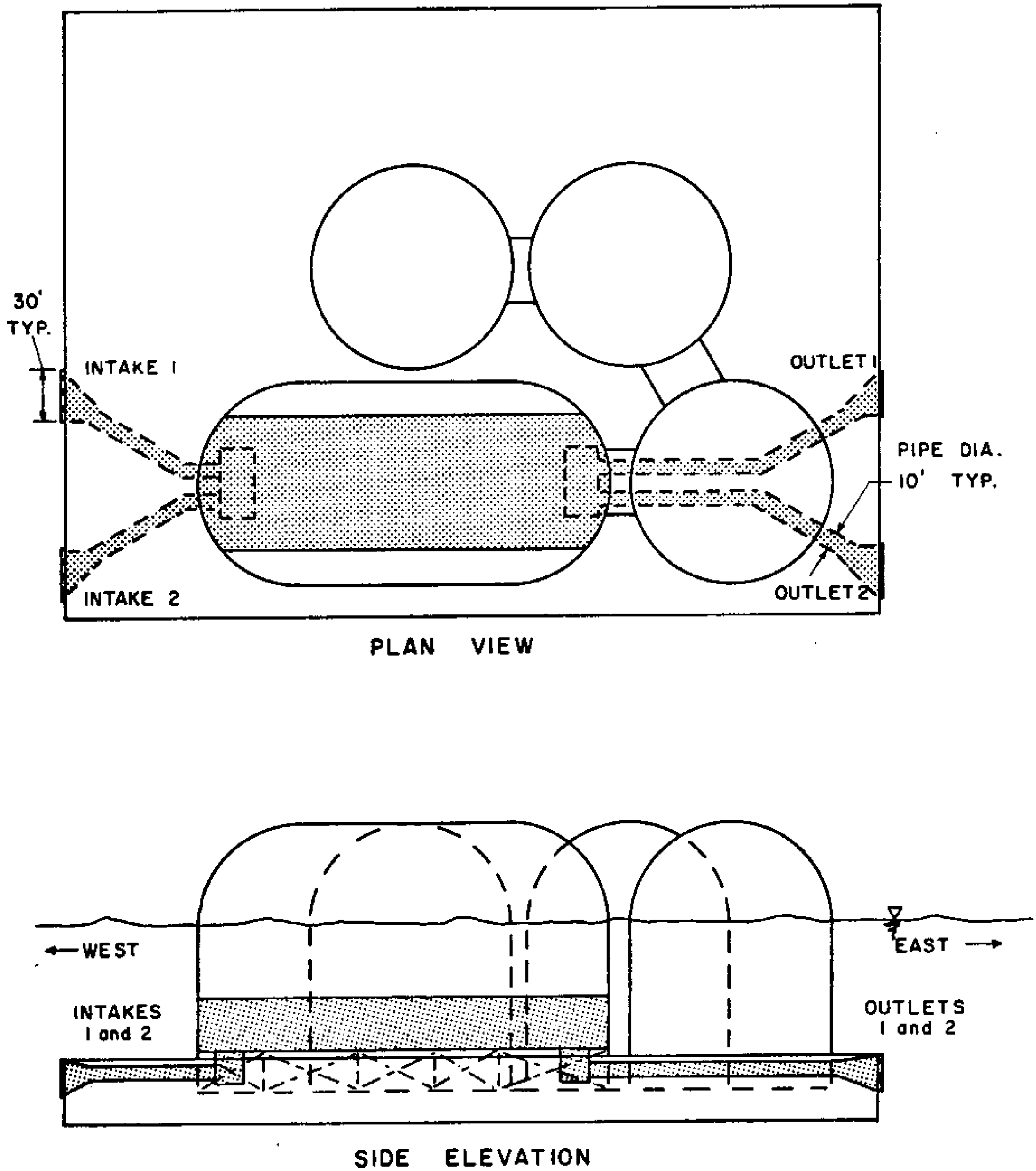


Figure 4-16. Sea water cooling system.

of copper alloys, graphitization of cast iron, micro and macro organism attack, breakdown of protective corrosion resistant films on copper alloys in high velocity seawater, and galvanic attack in adjacent zones of high and low velocity seawater.

Some specific corrosion problems and materials which will provide protection against corrosion are:

1. Corrosion of materials in the marine atmosphere

   All proposed building enclosures are concrete. Where piping or other metallic structures are required, 308 or 316 stainless steels, cupra-nickels, or common steels with high quality marine paint systems are recommended.

2. Corrosion of materials near the air-water interface

   High strength concrete is the material recommended for all structures which penetrate the water surface. This insures a minimum of wave erosion, no requirement for a galvanic protection in this zone, and a minimum of crevices.

3. Subsurface corrosion

   Submerged concrete surfaces would be subject to fouling by biological organisms and surface roughness would be increased. However, degradation of concrete due to fouling is minimal and the increase in surface drag is not critical. As in the surface zone, concrete will not promote galvanic action and resultant loss of materials in the anodic areas.

   The seawater cooling system would use 70-30 or 90-10 copper-nickel alloy for piping and water box materials. These alloys retain the high anti-fouling characteristics of copper while being highly durable, easy to fabricate and readily weldable. Most required marine hardware items are now available in a cupra-nickel alloy and their extensive use diminishes galvanic corrosion problems.

   All valve bodies would be M-Bronze. The stems, discs and seats of the valves would be of Monel. Secondary choices for trim, in order of preference, are Incoloy 825, Type 316 stainless and Type 304 stainless. This ordering is based upon merit regarding high velocity turbulence, resistance to crevice corrosion, galvanic effects, resistance to fouling by organisms and cost.

Any steel structure exposed to the subsurface environment must be protected against galvanic action by a cathodic protection system. The expected corrosion rate of unprotected steel surfaces is 0.005 inches per year, with localized pitting to depths of 0.015 inches per year. These effects can be minimized through use of properly designed impressed current systems.

4. Protection at and below the sea floor

At the sand or mud line, differential aeration cells develop and steel below this line becomes anodic to steel above it. Sulfate reducing bacteria in the marine silts can aggravate this action. Rate of corrosion can be increased by a factor of two over that experienced in open water. To avoid such problems all exposed subsurface structural surfaces should be concrete.

At the sea floor, some sand movement will undoubtedly take place and could cause abrasion of the concrete. Sharp corners should be avoided to minimize breaking and spalling.

5. Corrosion of reinforcement steel

Corrosion of embedded reinforcing steel in concrete structures located in the ocean is a serious problem. The deterioration rate of embedded steel is often much higher than that of exposed metals which form positive oxide films. In the splash zone, salt is deposited in the concrete in varying concentrations resulting in the creation of electrochemical cells and corrosion of the reinforcing steel. The corrosion products cause localized expansion and spalling of the protective concrete cover. Cloride concentrations in the mixing and curing waters must be kept to a minimum to inhibit the creation of such concentration cells during construction. Use of stainless steels for a reinforcing or prestressing material is not acceptable due to their high susceptibility to crevice corrosion (differential aeration cells). The most acceptable practice at present seems to be to utilize galvanized steel fittings and reinforcing steel and portland cement in the concrete mix. The portland cement creates deposits of calcium hydroxide during hydration which in turn maintain the internal pH at 13 to 14. Under such a condition, the reinforcing steel is covered by a thin protective oxide film. The efficient exclusion of chlorides (which tend to reduce the pH) oxygen and carbon dioxide by dense, impermeable and thick coatings of portland cement dictates its extensive use in the construction of the barge and plant enclosure buildings.

4.7 FOULING CONSIDERATIONS

The problems associated with the use of brackish water as a coolant in circulating-water power systems have been long standing nuisances. In past installations a solution has been to ignore fouling until plant efficiency dropped below an acceptable level, then shut down the plant and mechanically clean the system. In the underwater installation proposed, this alternative would be too time consuming and expensive to consider.

The main types of fouling which primarily affect the efficiency of the plant are: a) marine fouling in the intake-outflow structures which may seriously impede flows and result in decreased pumping efficiency, b) sliming in condenser tubes, thus decreasing the efficiency of heat transfer. A technique or combination of techniques is necessary to control both of these problems since the control of one does not necessarily follow directly from the control of the other.

Four methods of dealing with fouling have found varying degree of acceptance. They are:

1) Manual cleaning.
2) Heating the cooling water.
3) Introduction of poison-chlorination.
4) Various combinations of the foregoing.

The primary fouling organisms of interest include; Algae and slimes, Sea Anemones, Barnacles, and Mussels. All of these organisms gain entrance as microscopic larvae and must find a point of attachment within a short period of time. Consequently any effective control mechanism must take into account the life history and breeding cycle of the problem organism so as to ascertain the proper period for application of countermeasures. Fox et al[46] report that the anti-fouling measures should be applied while the invading populations are changing from the free swimming larval stages to the sessile attached bivalve stages.

Of the four methods of control mentioned, manual cleaning is unacceptable under the conditions of the proposed plant. Anderson and Richards[47] found that continuous chlorination of the intake water was generally effective against sliming organisms such as algae, but there was presented the problem of higher residuals of chlorine required for barnacle and mussel control. These high residuals of chlorine required for control had a corrosive effect on the metal portions of the system, especially those with copper alloys. The use of intermittent heating of intake water coupled with reverse flushing was found to be effective in controlling all forms of fouling organisms. Chadwick, Clark and Fox[36] have shown that control of Mytilus, a genus of mussels responsible for much of the fouling, could be accomplished by recycling heated water through the intake system for varying lengths of time, depending on water temperature. Figure 4-17 shown below indicates the thermal death time for the mussel Mytilus, since it is the most troublesome and resistant to elevated temperatures, therefore any temperature which proves lethal for Mytilus will also be fatal for other organisms.[36]

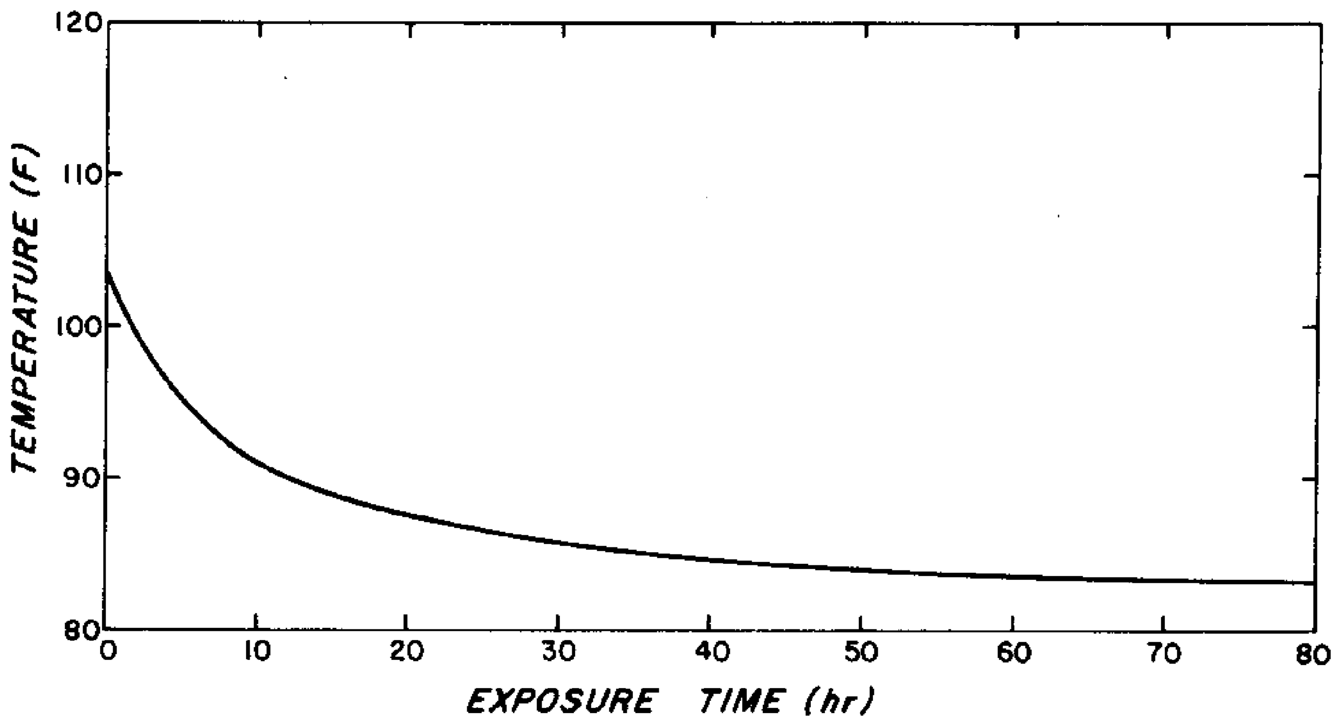


Figure 4-17. Lethal temperature vs. exposure time for the mussel Mytilus.

Chadwick et al[36] studied the control problems at Redondo Steam Station of the Southern California Edison Company, and concluded that the most effective way of dealing with fouling was by a combination of low level chlorination coupled with periodic reversal of flow through the structure so as to kill, dislodge, and flush out attached organisms such as mussels and barnacles. Similar methods have been chosen at various California sites including the nuclear generating plant at San Onofre. The intake design is not complicated (see section 4.1.2) and should include stations for injecting and monitoring chlorine residuals.

The flushing time required varies with temperature. Southern California installations flush with sea water warmed to 100-106° F and discharged through the intake tunnel for one hour, or water heated to 95°F and discharged through the intake tunnel for 7 hours tri-weekly, during the season of potential fouling.[46]

The operating cost of thermal treatment is relatively small when compared to chlorination. It was shown that the cost to clean the Redondo Plant is about $800 annually due to the loss of condenser vacuum and attendant higher fuel costs. The annual cost of the chlorine required to accomplish the equivalent protection was estimated to be $50,000.[36] These are estimates for a small plant, and a plant as large as the one under consideration would show an even greater disparity between the two methods.

The thermal flushing capability may be initially incorporated in the design of the circulation system so as to affect plant operations as little as possible. In addition to the thermal flushing apparatus, it is important to design structures which enhance the effectiveness of any control system by excluding rough surfaces which allow organisms to become attached, or stagnant areas in the flow tunnels which might harbor embryos, reduce the effects of current scouring, or impede the diffusion of chlorine. The establishment of mature fouling organisms in a low energy environment provides further shelter to encourage settlement by more fouling organisms.

The conclusions which may be drawn from these examples are that it is advisable to use a continuous low level chlorination treatment to control sliming organisms, while at the same time taking into consideration the effective, low cost thermal flushing methods available for control of attached fouling organisms which decrease plant efficiency by restricting the flow through the intake-outflow structures.

4.8 POWER TRANSMISSION SYSTEM

One of the most challenging problems associated with offshore power plants is the transmission of the electrical power to the mainland. In this section, three major components of a proposed transmission system are discussed; the switchgear at the site, the transmission cables, and the shore terminal.

4.8.1 Site Switchgear

The site switchgear includes all of the major pieces of equipment required to transform the generated voltage (18KV) up to transmission voltage (345KV), to protect the generator from line faults, and to give the system sufficient flexibility to permit continued operation with any single component out of commission. A simplified schematic of the transmission system is illustrated in Figure 4-18.[6]

Each component and all busses are capable of carrying the full power output of the generator. The three-phase power would normally be transmitted over two parallel transmission lines with the bus-tie breaker open. In the event of a fault in one line, only half of the load would be dropped, and the remaining cable could then be utilized to carry the full load. This redundancy in transmission paths also means that each path is only operated at one half of its design capacity, with an attendant improvement in reliability of all components in the system.

Two types of circuit breakers could be employed in the switchgear system. Gas-filled (sulfur hexaflouride) high voltage breakers could be utilized for current interruption.[48] Two of these breakers in series in each line would provide adequate fault protection for the generating system. Another pair of breakers at the shore terminal would protect the power grid from potential cable faults. In addition, numerous standard mechanical breakers would be installed to provide back-up fault protection and to allow various components to be isolated for maintenance and repair.

4.8.2 Transmission Cables

There are two basic approaches to the problem of transmitting large amounts of electrical power -- one employs standard overhead high voltage transmission lines, while the other utilizes some kind of underground (or underwater) cable. The overhead transmission line approach was ruled out for a multitude of reasons, including the feasibility and cost of supporting the lines over 2 1/2 miles of open ocean water, the vulnerability of the lines and supports to damage by offshore weather conditions, and recognition of the fact that overhead transmission lines to the seashore would be very undesirable from an aesthetic viewpoint. Underground cables, on the other hand, are seeing increased

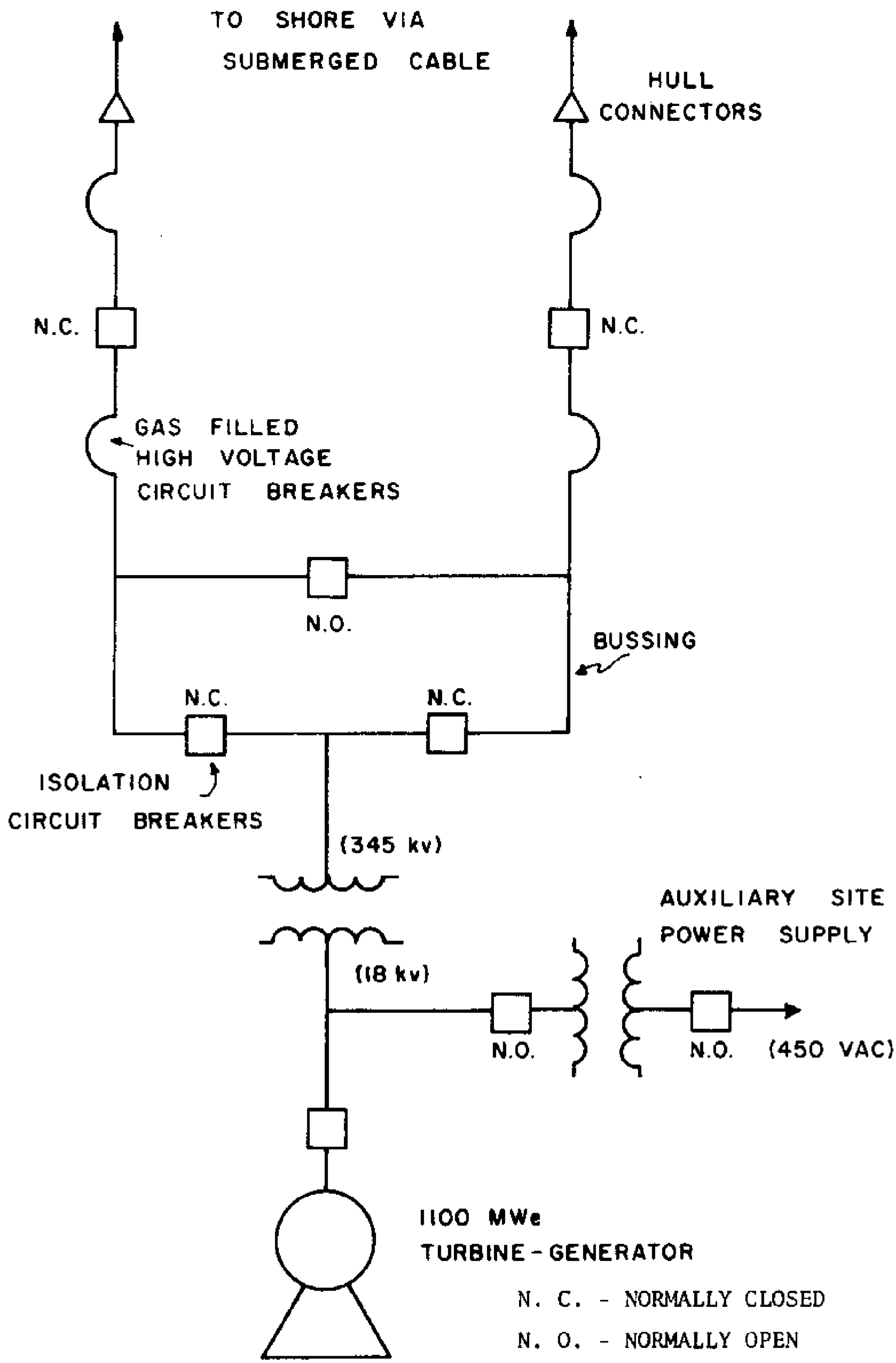


Figure 4-18. Power transmission system schematic.

application in large cities, and the state-of-the-art of cable development is sufficiently advanced to make a cable installation feasible.

There are two possible cable systems which could be considered. The first is a high-pressure, oil-filled pipe-type cable. This type of cable has been successfully used by Southern California Edison to provide power to offshore drilling islands in Long Beach Harbor, by Enfield-Standard Power Cables Ltd. of Middlesex, England for power cables under the Thames Estuary, and by Consolidated Edison of New York to transmit power under the Narrows Crossing.[49] The cable consists of three single-phase, oil-paper insulated conductors contained in a pressurized, oil-filled pipe. The steel pipe is coated with mastic and concrete and laid in a shallow (4'- 6') trench on the sea floor. The three conductors are then pulled through the pipe. However, since this application involves a longer cable than any pulls attempted to date, new methods of reducing friction would probably have to be developed. This system would require an oil pressure system with a pumping plant at either the site or the shore terminal.

A second possibility would be to use cryogenic cables.[50] Nitrogen cooled near-superconducting cables are showing increasing promise for underground installations, although they probably won't be commercially available until 1975. A typical cable would contain three tubular anodized aluminum conductors filled with liquid nitrogen coolant. The conductors are shielded and supported by dielectric spacers in a corrosion protected, vacuum-tight steel pipe.[51] While this type of cable might permit economic transmission of full power at generated voltage (thus eliminating transformers, etc), laying of the cable filled pipe would definitely be a technological challenge. This system would require a cooling station (liquid nitrogen refrigeration unit), which would probably be located at the site.

The trenches containing the power cables would also contain any other umbilicals to the shore, such as the 33KV site power cable, communications lines, etc.

4.8.3 Shore Terminal

The shore terminal provides an interface between the submerged cable and the power grid. Gas filled breakers similar to those utilized at the site would provide fault protection. The terminal could either connect directly to a 345KV grid, or the voltage could be raised to 500KV, depending on which voltage was compatible with the local grid. The transformer and breakers for the 33KV line to the site would also be included in this facility. As mentioned earlier, the shore terminal might include an oil pressure system, or possibly a nitrogen cooling system, depending on the method of transmission. Figure 4-19 illustrates a possible shore terminal configuration.

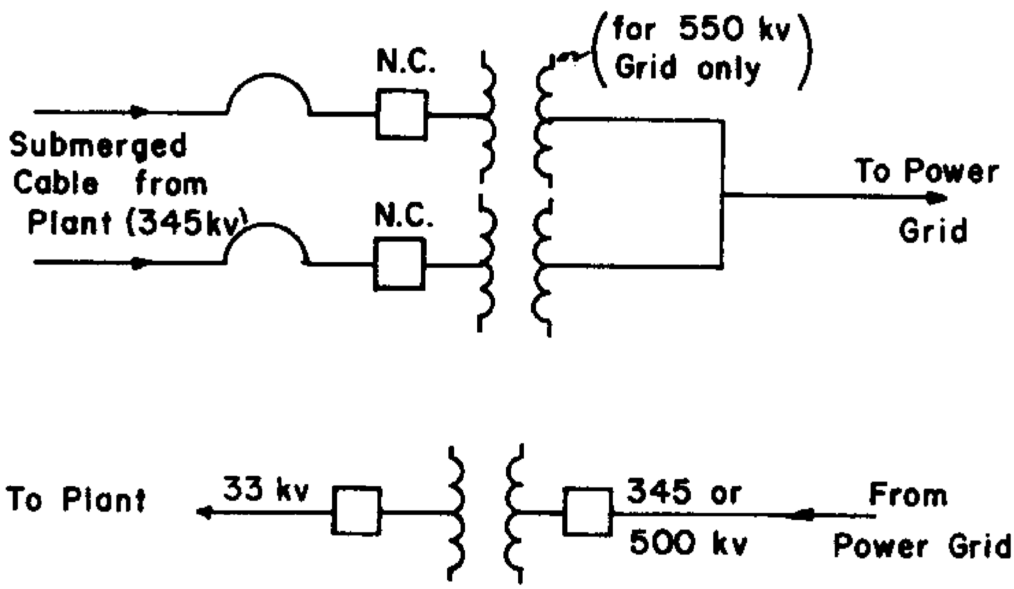


Figure 4-19. Shore terminal configuration.

5. Wind and Wave Loadings

5.1 DESIGN WAVE

A design wave is typically determined on the basis of the characteristic 50 or 100 year storm appropriate for the proposed structure site. The maximum significant wave height of 59 feet calculated in Section 3.0 is based on extreme swell and sea recorded off the Oregon Coast for a 3 year period. To verify that these data represent the nature of wave characteristic of a 100 year design storm, a major North Pacific storm which occurred in November of 1969 was chosen as the design storm. (Figure 5-1)

The data available on this storm are used to calculate the significant period and wave height of the deep water generated waves which initially arrived at the Oregon Coast in the form of long period swell. The effects of refraction, shoaling and friction-percolation are considered in determining the resultant swell height at the proposed plant site. As the storm propogated eastward over the continental shelf and past the Oregon and Washington coastlines, shallow water waves were generated. The characteristics of these 'seas' are calculated and compared to the long period swell. The most severe of the two wave types is then used as the basis for calculating a maximum design wave.

Bretschneider in 1953 developed empirical relationships between the wave energy index (RΔP) and the storm generated deep water significant wave height and period: (52, 53, 54)

$$H_S = 16.5\, \ell\, \frac{R\Delta P}{100} \left[1 + \frac{0.208\, \alpha\, V_f}{\sqrt{U_R}} \right] \quad . \; . \; . \; . \qquad (1)$$

$$T_S = 8.6\, \ell\, \frac{R\Delta P}{200} \left[1 + \frac{0.104\, \alpha\, V_f}{\sqrt{U_R}} \right] \quad . \; . \; . \; . \qquad (2)$$

where:

H_S = deep water significant wave height (ft.)

T_S = deep water significant wave period (ft.)

R = radius of maximum wind (nautical miles)

ΔP = reduction in atmospheric pressure from normal at the storm center (in. of mercury)

V_f = forward speed of the hurricane (knots)

α = dimensionless constant used to include the effects of increased wind speeds due to the forward speed of the storm. For slowly moving storms, $\alpha \sim 1.0$.

U_R = maximum sustained wind speed calculated for 30 feet above the mean sea surface at radius R (knots)

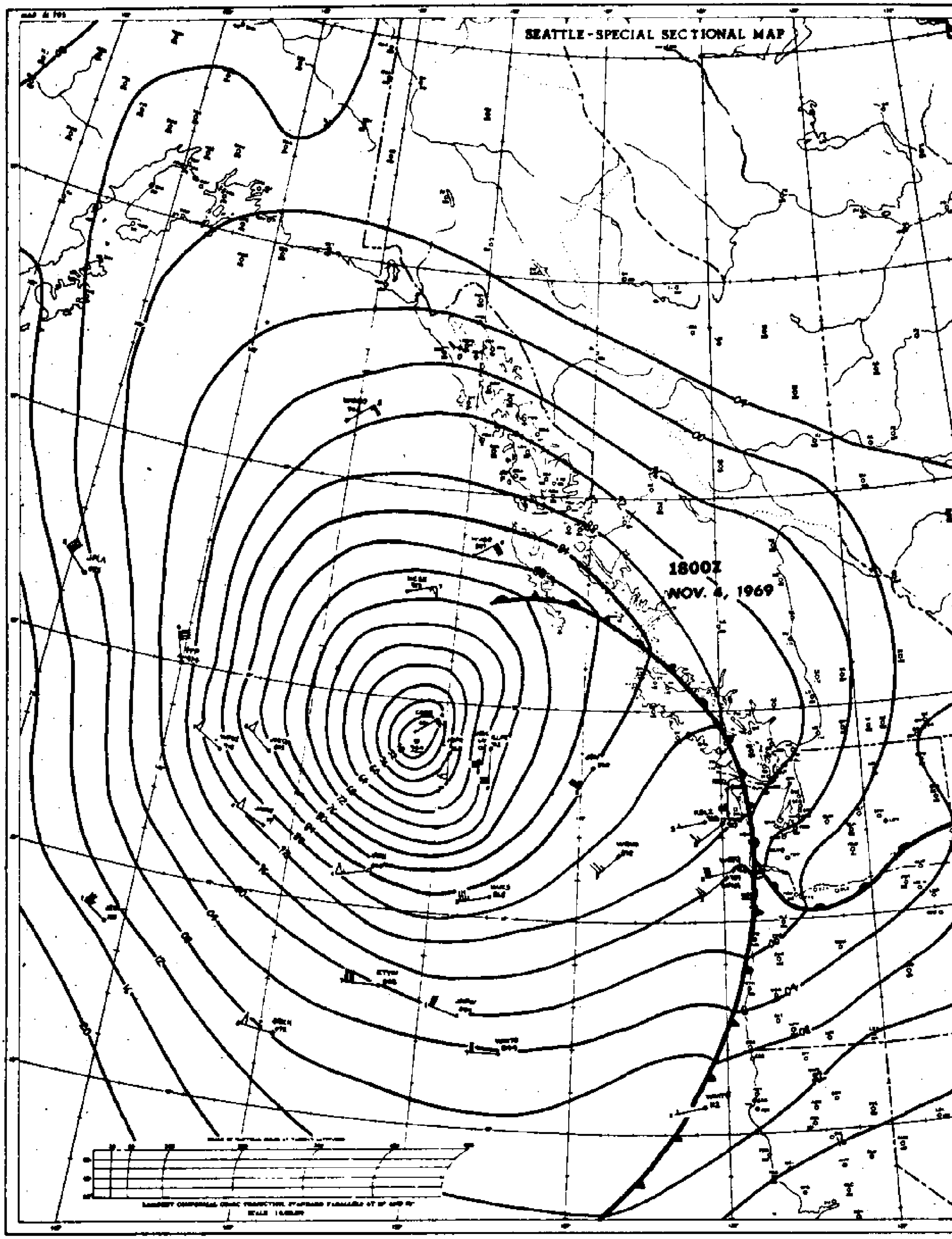


Figure 5-1. Design storm surface synoptic chart for 1800Z, 4 November 1969

U_{max} = maximum gradient wind speed 30 feet above the mean sea surface (knots)

$$U_R = 0.865\ U_{max} + 0.5\ V_f \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

$$U_{max} = 0.868\ [73\ (\Delta P)^{1/2} - R(0.575\ f)] \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (4)$$

$f = 2\ \omega \sin \phi$ = Coriolis parameter (0.378 for $\phi = 45°$)

ϕ = latitude (degrees)

ω = angular velocity of the earth (0.2675 radians/sec.)

Equations (1) and (2) are valid only if the forward speed of the storm is less than the critical speed given by:

$$V_{Cr} = 13.1\ \ell^{\frac{R\Delta P}{200}} \left[1 - \frac{1.36\ \alpha\ \ell^{\frac{R\Delta P}{200}}}{\sqrt{U_R}}\right]^{-1} \quad . \; . \; . \; . \; . \; . \; . \quad (5)$$

For the storm in Figure 5-1, the maximum wind radius R is 40 nautical miles, ΔP is 2.06 in. of mercury (70 mb.) and V_f = 25 knots.

$$U_{max} = 0.868\ [\ 73\ (2.06)^{1/2} - (40)\ (0.575)\ (0.378)\ ]$$

$$U_{max} = 0.868\ [\ 104.9 - 8.7\ ] = 96.2 \text{ knots}$$

$$U_R = (0.865)\ (96.2) + (0.5)\ (25) = 95.8 \text{ knots}$$

The energy index exponents are

$$\frac{R\Delta P}{100} = \frac{(40)(2.06)}{100} = 0.824$$

$$\frac{R\Delta P}{200} = 0.412$$

The corresponding deep water significant wave height and period are:

$$H_S = 16.5\ \ell^{0.824} \left[\ 1 + \frac{(0.208)(1.0)(25)}{\sqrt{95.8}}\ \right]$$

$$H_S = (16.5)(2.28)\ [\ 1 + 0.53] = 57.5 \text{ feet}$$

$$T_S = 8.6\ \ell^{\,0.412} \left[1 + \frac{(0.104)(1.0)(25)}{\sqrt{95.8}} \right]$$

$$T_S = (8.6)(1.51)\quad [1 + 0.266] = 16.5 \text{ seconds}$$

$$V_{Cr} = 13.1\ \ell^{\,0.412} \left[1 - \frac{(1.36)(1.0)\ \ell^{\,0.412}}{\sqrt{95.8}} \right]^{-1}$$

$$V_{Cr} = (13.1)(1.51)(1.27) = 25.2 \text{ knots}$$

Since the storm front is moving forward at approximately the critical speed, the conditions for maximum wave generation exist. If this speed had been exceeded, the significant wave height and period would have been significantly decreased.

The effective fetch length is:[54]

$$F_e = \left[\frac{H_S}{0.555\ U_R} \right]^2 \qquad (6)$$

$$F_e = \left[\frac{57.5}{(0.0555)(95.8)}\right]^2 = 116 \text{ nautical miles}$$

When waves propagate into shallow water they are affected by refraction, shoaling and bottom losses associated with friction and percolation.

$$H_S' = H_S\ [K_r\ K_S\ K_{fp}] \qquad (7)$$

where

K_r = refraction coefficient = $\left[\frac{b_o}{b} \right]^{1/2}$

b_o = distance between deep water orthogonals

b = distance between shallow water orthogonals

K_S = shoaling coefficient = $\left[\left(1 + \frac{2kd}{\sinh 2kd} \right) \tanh pd\right]^{-1/2}$

k = wave number = $\frac{2\pi}{L}$

L = wave length

d = water depth

K_{fp} = friction - percolation coefficient = $\left(\frac{Pb}{P_o b_o}\right)^{1/2}$

P = power transmitted across a vertical section bd

For the purpose of this storm, it is assumed that the waves propagate from the West towards a North-South section of the Oregon coastline (Fig. 5-2). Where the continental shelf is reasonably smooth and refraction is low, K_r can be approximated as 1.0. The details of calculating the shoaling and frictional effects on the storm swell as it propagates from deep water to 150 feet is discussed in Reference 54. The results of such calculations are shown in Table 5-1. The most probable maximum waves can be calculated from

$$H_{max} = 0.707\ H_S \sqrt{\ell n \frac{N}{n}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

where

N = number of waves considered

n = 1 for the first most probable maximum wave,
2 for the next most probable etc.

The formula of $H_{max} = 1.86\ H_S$ is valid for n = 1 and N = 1000. However for a moving storm, the number of waves which will occur during the time it takes the radius of maximum wind to pass a point is:

$$N = \frac{t}{T_S}$$

where

$$t = \frac{R}{V_f} = \text{time for radius of maximum wind to pass a point (seconds)}$$

For this storm

$$N = \frac{(40)(3600)}{(25)(16.5)} = 348 \text{ waves}$$

and

$$H_{max} = 0.707 \sqrt{\ell n\ 348}\ H_S$$

$$H_{max} = 1.72\ H_S$$

The predicted most probable maximum waves at the proposed site due to the storm considered are 87.0 feet from trough to crest. As far as shoaling effects are concerned, the site is a best location in that the minimum value of the shoaling coefficient is in effect for a water depth of 175 feet for the 16.5 second waves. The friction percolation losses are not effective until water depths of the order of 200 feet, and hence will not significantly diminish the amplitude of the approaching waves.

5.2 GENERATION OF WIND WAVES NEAR SHORE

The previous calculations are relevant for a storm which is located a large distance from the site in question. Also of concern is the nature

Sta.	X	d_x	d_1	d_2	$\bar{d}_T$	F_e	H_S	T_S	$T_S^2/\bar{d}_T$	K_{S_1}	A	K_f	H_S'	F_e'	T_S'	$T_S'^2/d_2$	K_{S_2}	H_S'	N	H_{MAX}
S_0	16	500	633	508	570	116	57.5	16.5	0.475	0.980	0.021	1.00	57.5	116	16.5	0.54	0.97	56.7	348	97.5
S_1	14	380	508	388	448	116	57.5	16.5	0.610	0.955	0.033	1.00	57.5	116	16.5	0.70	0.94	54.0	348	93.0
S_2	12	330	388	338	363	116	57.5	16.5	0.750	0.935	0.049	1.00	57.5	116	16.5	0.80	0.932	53.5	348	92.0
S_3	10	300	338	308	323	116	57.5	16.5	0.840	0.928	0.062	1.00	57.5	116	16.5	0.88	0.925	53.2	348	91.5
S_4	8	260	308	268	288	116	57.5	16.5	0.945	0.922	0.077	1.00	57.5	116	16.5	0.94	0.922	53.0	348	91.0
S_5	6	230	268	238	253	116	57.5	16.5	1.080	0.920	0.101	0.99	57.0	114	16.1	1.08	0.920	52.5	358	89.5
S_6	4	175	238	183	210	116	57.5	16.5	1.30	0.915	0.145	0.99	57.0	114	16.1	1.42	0.920	52.5	358	89.5
S_7	2	150	183	158	170	115	56.7	16.0	1.51	0.923	0.221	0.97	55.0	110	15.7	1.56	0.925	51.0	368	87.0

Table 5-I Calculations of H_{max}

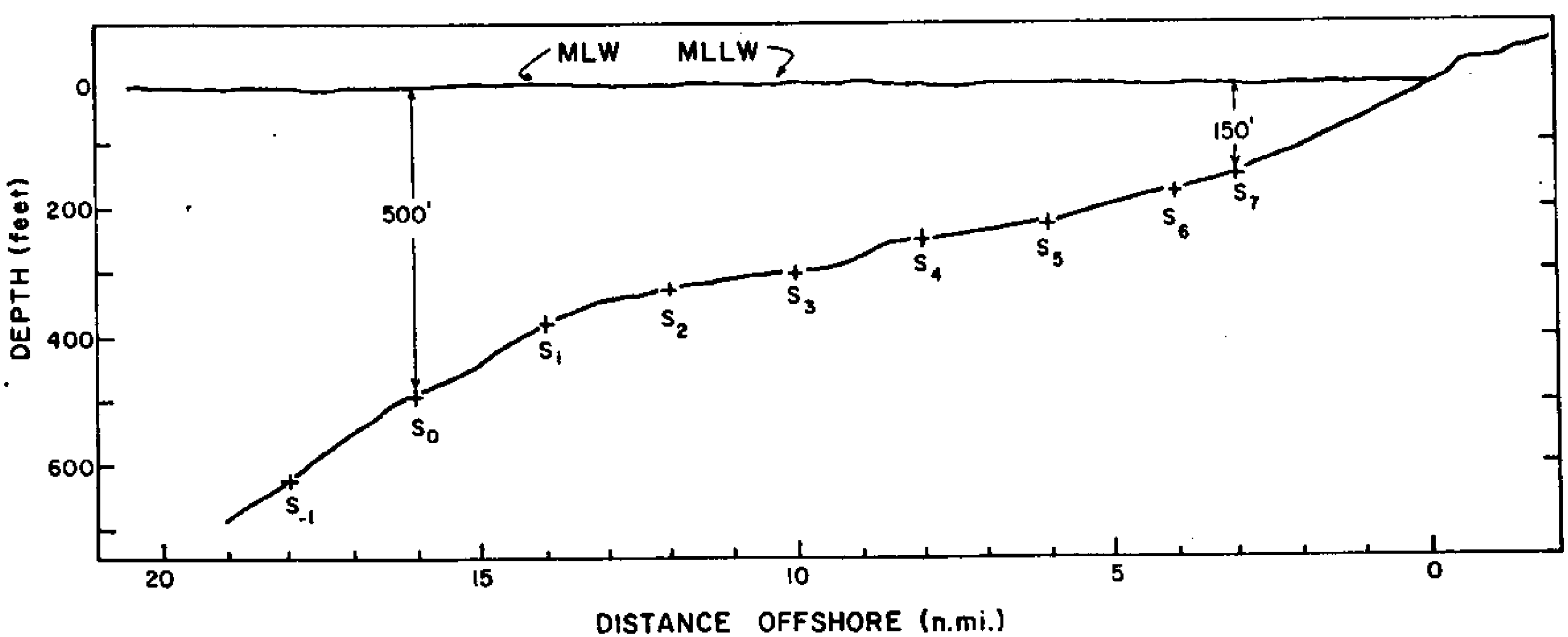


Figure 5-2 Offshore profile of Oregon coastline.

of the wind generated waves which occur as a storm passes over the plant site. As the radius of maximum wind passes the site, the significant wave height will be 57.5 feet and the corresponding most probable maximum wave height will be 99.0 feet.

5.3 STORM SURGE

The maximum storm surge recorded at Newport Bay, Oregon is 3.8 feet.

5.4 DESIGN WIND SPEED

A wind speed must be specified as being over land or water, calculated from the surface pressure gradient; whether it is the fastest mile, one or ten minute average, etc. To calculate wind forces on a structure, not only the average sustained wind speeds, but the effects of gusts must be considered. The gust factor is defined as that factor which must be applied to the sustained wind speed to obtain the gust speed.

$$U_R = 95.8 \text{ knots}$$

$$\underline{V_f = 25.0 \text{ knots}}$$

$$U_{10} = 120.8 \text{ knots} = 204 \text{ ft/sec}$$

The length of gust considered effective on the structure is 3 to 5 times the length of the structure.

$$\ell = 5(500) = 2500 \text{ feet}$$

The duration of the gust will then be approximately (assuming $G_{avg.} \approx 1.4 \times 204 = 286$ ft/sec.)

$$t = \frac{\ell}{U_G} = \frac{2500}{286} \cong 8.7 \text{ seconds} \quad \ldots\ldots\ldots\ldots \quad (10)$$

Figure 5-3 indicates that an appropriate gust factor is 1.35 using the center dashed line for a 10-minute $\overline{U}$.* Thus the design wind speed is

$$U_{Gust} = (1.35)(120.8 \text{ knots})$$

$$U_{Gust} = 163 \text{ knots}$$

* This F_{10} = 1.35 compares with F_{10} = 1.45 for Lake Okeechobee where U_{10} = 10 meter wind speed, 10 minute average. Factory Mutual Insurance Company recommends F_{10} = 1.50 where U_{10} = fastest mile of wind speed. (ref.55)

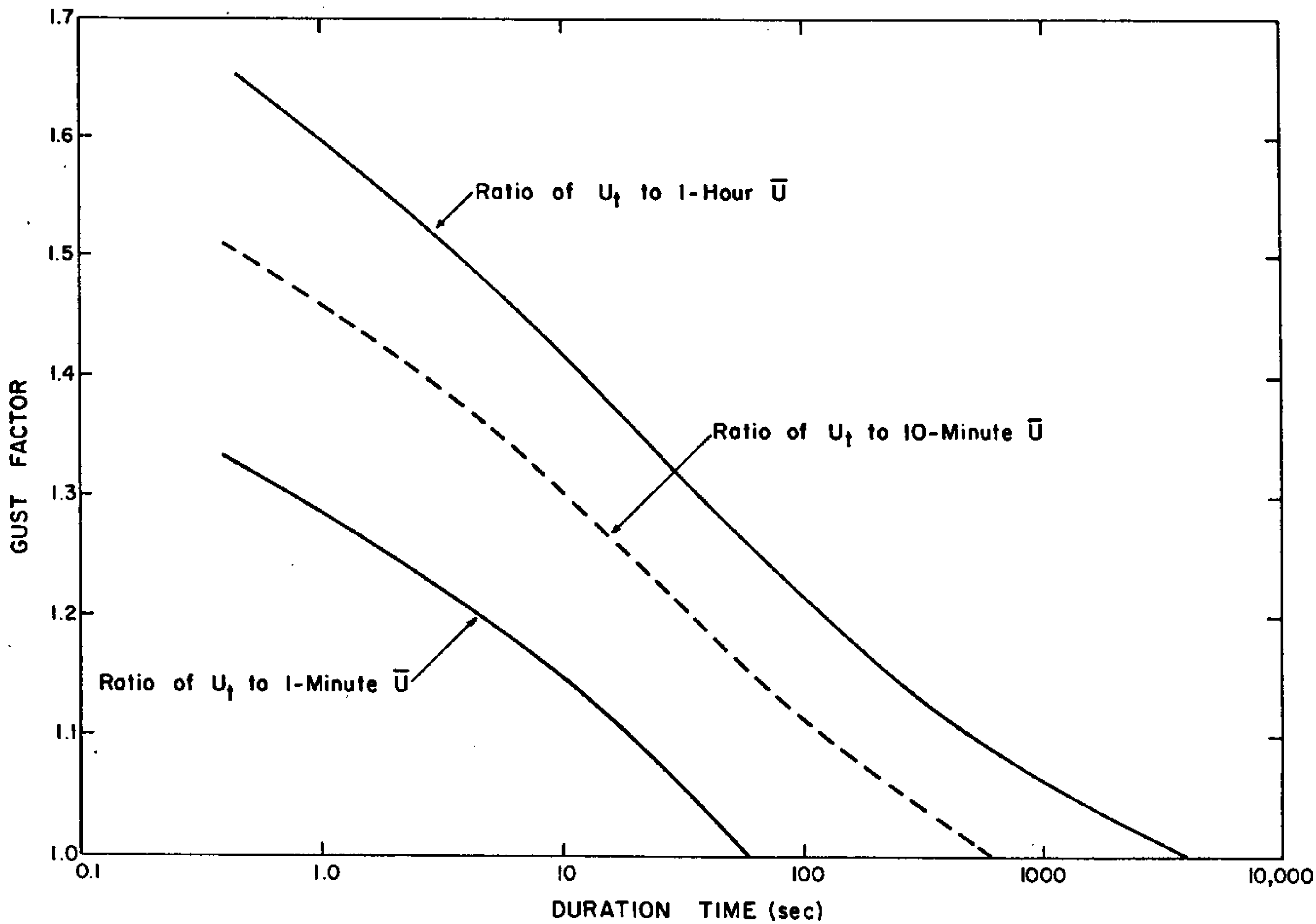


Figure 5-3. Wind ratios for various wind durations (from

5.5 CREST ELEVATION AND WAVE PROFILE

Except for very small wave steepness, where Airy Theory applies, there is no theory which adequately describes wave profiles in waters of intermediate depths ($0.08 < d/T^2 < 2.0$). Using the Airy Wave Theory, Stokes Struik Theory, Solitary Wave Theory and the Michell and Havlock Theories, Bretschneider and Reid prepared Figures 5-4 and 5-5, generalized graphs of crest elevations for all waves[4].

For this design storm and water depth:

$$\frac{d}{T^2} = \frac{150}{(16.5)^2} = 0.55$$

$$\frac{H}{T^2} = \frac{99}{272} = 0.364$$

For these values Figure 5.4 gives $\eta_o/H = 0.725$

$\eta_o = (0.725)(99) = 72.0$ feet elevation of the crest above still water level.

Ratios of elevation at a given phase angle to the maximum elevation can be determined from Figure 5-5. The resultant wave profile is shown in Figure 5-6.

5.6 MAXIMUM WAVE FORCES

$$F_{DM} = \frac{1}{2} \rho C_D K_{DM} H^2 D \quad \ldots \quad (1)$$

$$F_{IM} = \frac{1}{2} \rho C_M K_{IM} D^2 H \quad \ldots \quad (2)$$

where

F_{DM} = maximum value of the integrated horizontal drag force beneath the wave crest.

F_{IM} = maximum value of the integrated horizontal inertial force which occurs at some point between the wave crest and one quarter of the wave length.

C_M, C_D = coefficients of mass and drag respectively.

K_{DM} = drag force factor

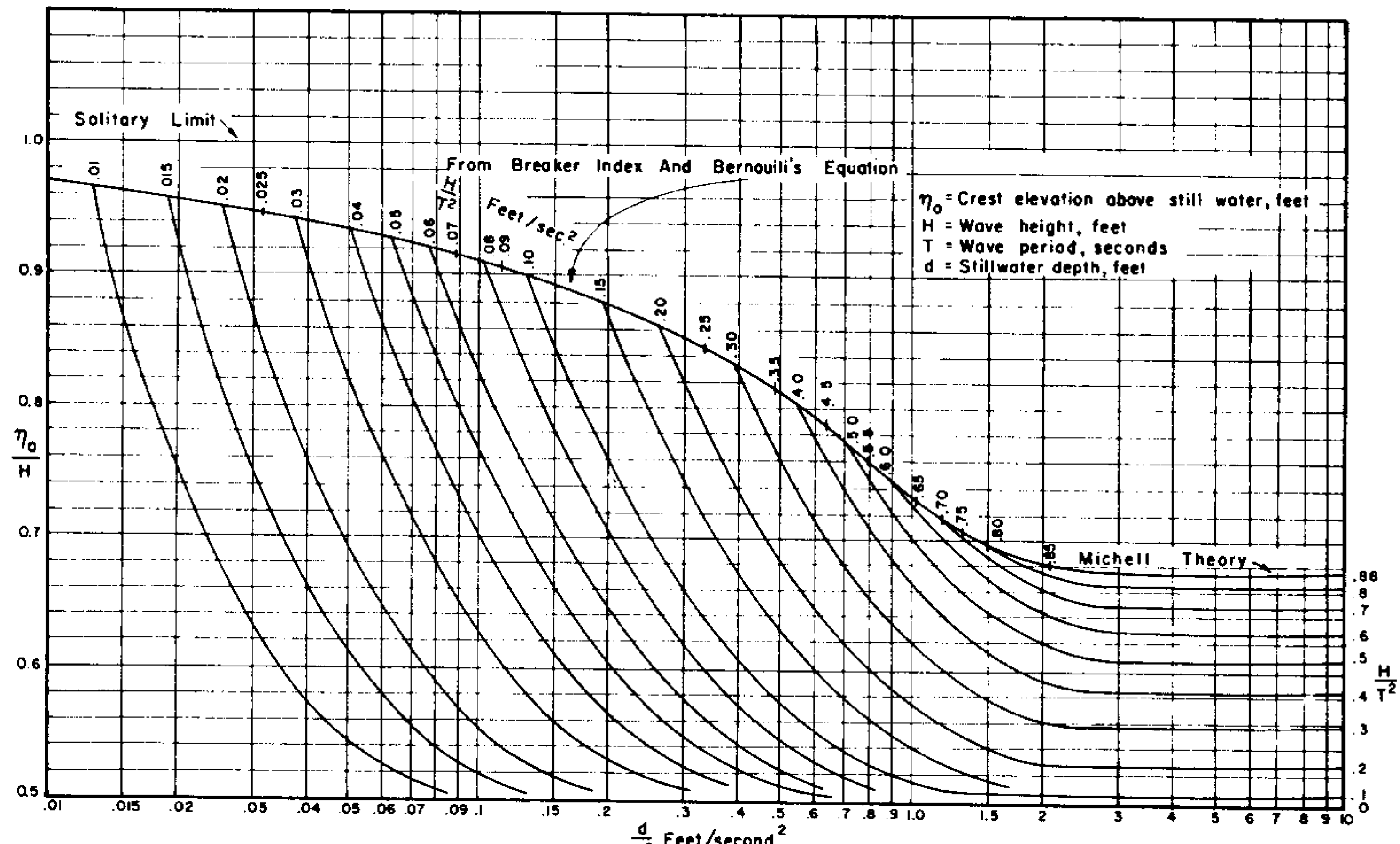


Figure 5-4. Ratio of crest elevation above still water to wave height (from U.S. Army Corps of Engineers, 54).

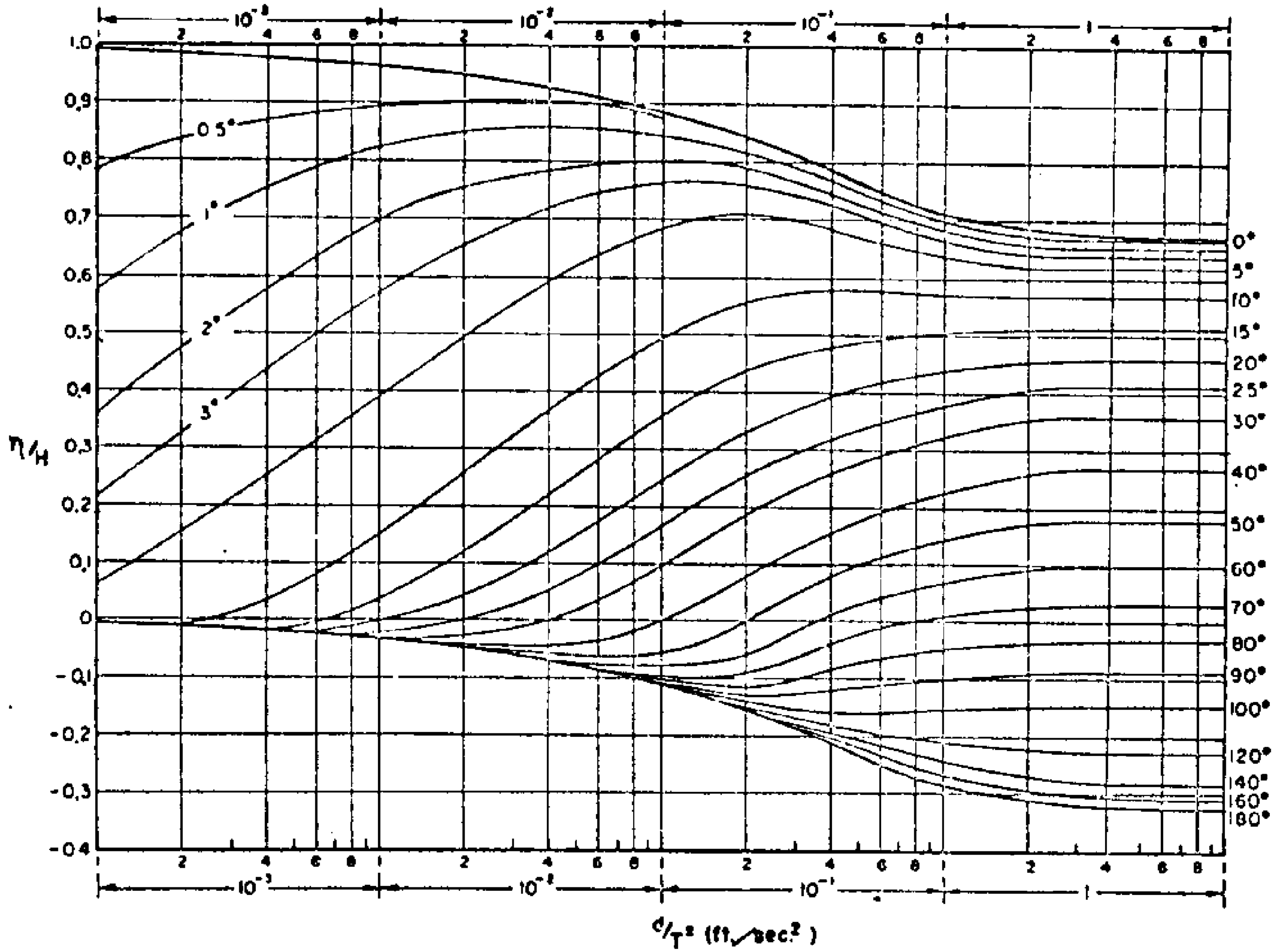


Figure 5-5. Relative surface elevation versus relative depth for the maxiumum wave (from Reid and Bretschneider, 55).

θ	η_o/H	η_o
0	0.79	73.0
5	0.66	65.5
10	0.57	56.5
15	0.48	47.5
20	0.41	40.6
25	0.35	34.7
30	0.28	27.8
40	0.19	18.8
50	0.10	9.9
60	0.03	2.98
70	-0.02	-1.98
80	-0.07	-6.92
90	-0.12	-11.2
100	-0.15	-14.8
120	-0.20	-19.8
140	-0.21	-20.0
160	-0.23	-22.8
180	-0.25	-24.7

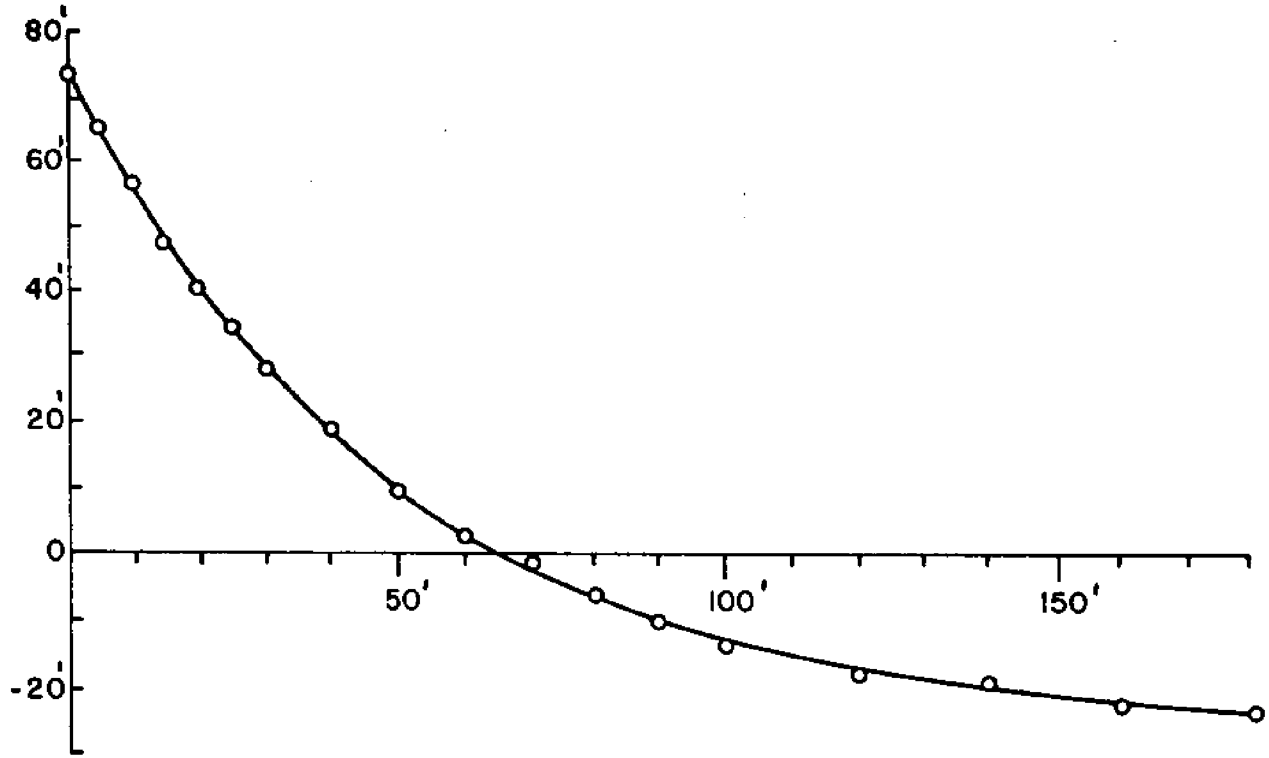


Figure 5-6. Design wave profile.

K_{IM} = inertial force factor

D = structure diameter

Choosing

$$C_D = 0.63$$

$$C_M = 2.0$$

From figures 5-7 and 5-8

$$K_{DM} = 19$$

$$K_{IM} = 20$$

$$F_{DM} = (\tfrac{1}{2}\,\rho)\ (0.63)(19)(92)^2(75)$$

$$F_{DM} = 7650 \text{ KIPS}$$

$$F_{IM} = (\tfrac{1}{2}\,\rho)\ (2.0)(20)(75)^2(92)$$

$$F_{IM} = 20{,}700 \text{ KIPS}$$

These loadings represent that expected on a single submerged cylinder. Any rigorous calculation would necessarily consider sheltering effects, but for purposes of approximation, the forces on each cylinder will be considered isolated.

$$F_{DM_{TOT}} = (4)(7650) = 30{,}600 \text{ KIPS}$$

$$F_{IM_{TOT}} = (4)(20{,}700) = 83{,}000 \text{ KIPS}$$

Due to its large dimensions, the major force component on the submerged barge can be considered inertial. For such a case, the barge dimensions are

$$L_1 = 600 \text{ ft.}$$

$$L_2 = 450 \text{ ft.}$$

$$L_3 = 50 \text{ ft.}$$

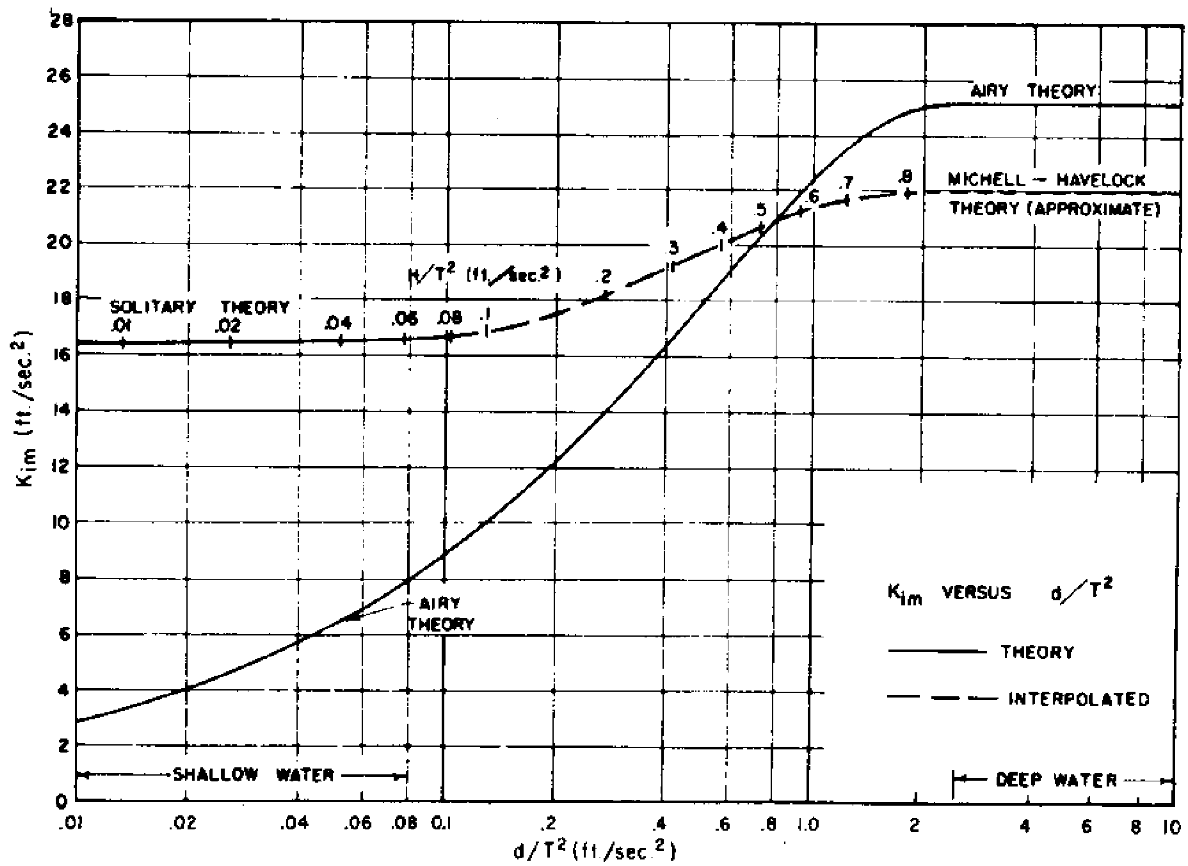


Figure 5-7. K_{im} versus d/T^2 (from U.S. Army Corps of Engineers, 54)

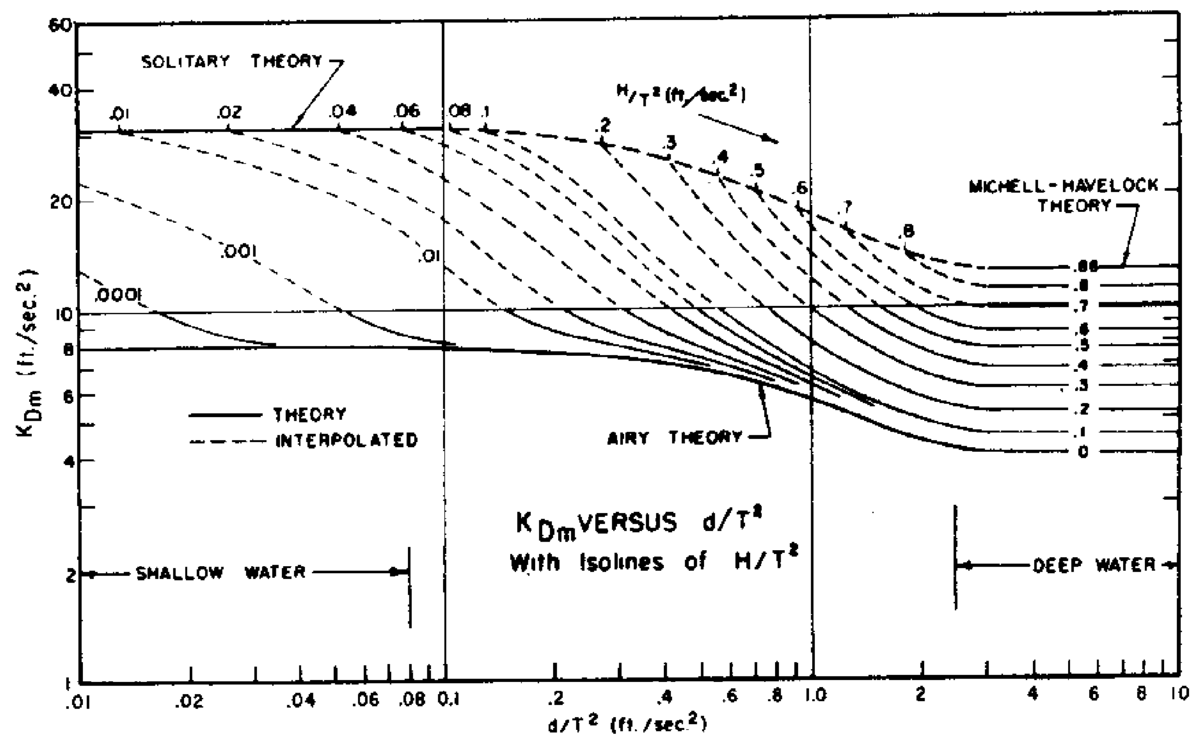


Figure 5-8. K_{Dm} versus d/T^2 (from U.S. Army Corps of Engineers, 54)

According to Bretschneider and Reid:

$$F_{max} = C_m\ L_2\ L_3\ K\ \omega\ H \sin \frac{\pi L_1}{L} \quad . . . . \quad (3)$$

where

$$K = \text{pressure factor} = \frac{(\Delta p)_c - (\Delta p)_t}{\omega H}$$

$$\omega = 64\ \text{lb/ft}^3$$

$$(\Delta p)_c = \text{pressure anomaly beneath the wave crest}$$

$$(\Delta p)_t = \text{pressure anomaly beneath the wave trough}$$

$$L = \text{wave length}$$

for

$$\frac{d}{T^2} = 0.55 \text{ and } \frac{H}{T^2} = 0.364,\ K = 0.58$$

and

$$\frac{L_A}{L_o} = 0.72 \text{ where}$$

$$L_a = \text{wave length for period T and depth d}$$

$$L_o = 5.12T^2 = \text{deep water wavelength}$$

$$\frac{L}{L_a} = 1.16$$

$$L_o = (5.12)(16.5)^2 = 1390\ \text{ft.}$$

$$L = (0.72)(1.16)(1390) = 1160\ \text{ft.}$$

$$\frac{L_1}{L} = \frac{600}{1160} = 0.508$$

$$\frac{\pi L_1}{L} = 1.59 \text{ radians} = 91.7°$$

$$F_{max} = (2.0)(450)(50)(0.58)(64.0)(99) \sin 91.7°$$

$$F_{max} = 166{,}000\ \text{KIPS}$$

5.7 MAXIMUM WIND FORCES

There is some variation in gust speed with elevation above the water's surface.[56]

$$\frac{G_z}{G_{10}} = \left(\frac{z}{10}\right)^m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

where

G_z = gust speed at elevation z

G_{10} = gust speed at 10 meters

z = elevation above the water's surface

m_1 = 1/7 recommended by ASCE Task Committee on Wind Forces for very high wind speeds over smooth surfaces

m_2 = 1/12 data of Sherlock and Deacon for high wind speeds

The calculation at hand is somewhat simplified by the fact that the mean height of the structure above MWL is 35 feet, so that G_{10} can be used directly.

$$F_D = C_D\, qA = C_D \frac{\rho U^2}{2} A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

where

C_D = drag coefficient

A = projected area

q = dynamic pressure ($0.0034\ U^2$ when U is expressed in knots)

$$F_D = \frac{(0.45)(0.0034)(163)^2\ (3.14)(75)^2\ \cancel{2}}{(\cancel{2})}$$

F_D = 720 KIPS applied at an elevation of 187.5 feet above the sea floor.

5.8 MAXIMUM OVERTURNING MOMENTS

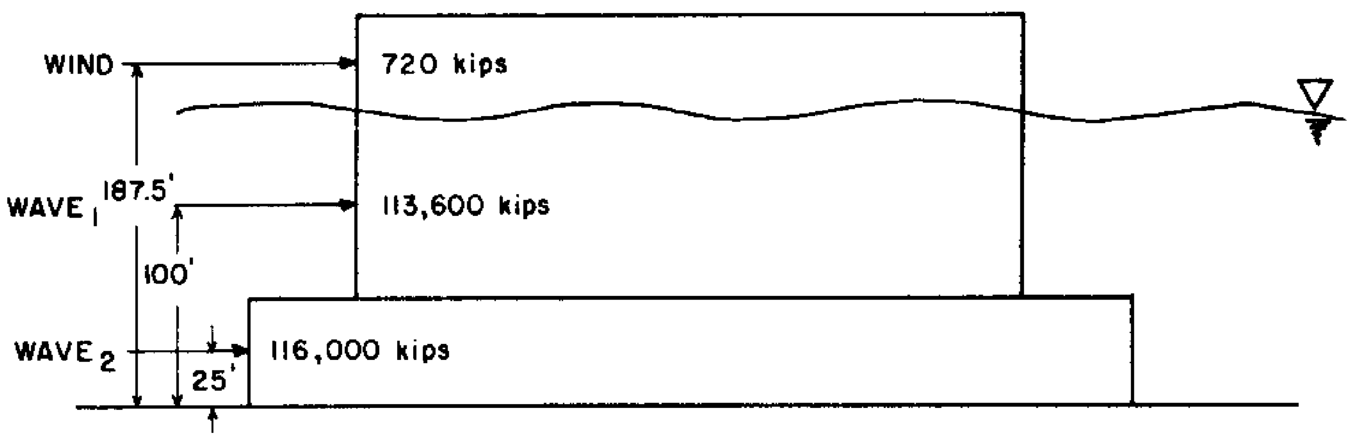


Figure 5-9. Maximum overturning moments.

$$\Sigma M = (720)(187.5) + (113{,}600)(100) + (166{,}000)(25)$$

$$= (1.35)(10^5) + (1.14)(10^7) + (4.15)(10^6)$$

$$\Sigma M = 15{,}685{,}000 \text{ ft. - KIPS}$$

6. Site Preparation and Foundation

In the placing of a nuclear power facility offshore, design of a suitable method of support and preparation of the particular site choosen will be perhaps the most challenging aspect of the entire project, and also the greatest uncertainty in terms of cost in dollars. For this study, one approach to placing the nuclear power facility is examined in detail, and several alternatives are suggested. The calculations presented are intended only to indicate the parameters to be considered in a complete design.

6.1 WATER LEVEL CHANGES

When the barge and hull are semi-submerged in 150 feet of water, it is estimated the net weight will be approximately 35,000 tons. (reference section 4.3.1) However, due to water level fluctuations, the water will not always be 150 feet in depth.

6.1.1 Changes Due To Tides

Recent data indicates the mean daily tidal range (MHHW to MLLW) to be 8.37 feet[57]. Also, the highest recorded tide has been 12.62 feet above MLLW, while the lowest recorded tide has been 3.15 feet below MLLW. The low tide will result in increased loads on the foundation, while high tides will reduce the loadings.

6.1.2 Changes Due To Tsunamis

Literature indicates that the height of even a very large tsunami in 150 feet of water will be quite small (less than 2 feet). The very large wave heights one often associates with tsunamis would be due to shoaling affects as the wave nears the shore.

6.1.3 Changes Due To Storm Waves

For a design storm wave, the maximum probable wave calculated in Chapter 5 will be utilized. Though this is perhaps a larger wave than would ever be observed at the structure, a wave of this magnitude is theoretically possible given that the design storm occurs, and thus it results in a conservative design margin. The design storm wave has a height of 99 feet and a period of 16.5 seconds. For the order of approximation required here, linear wave theory will be used to determine the average change in water level due to storm waves. Thus, the wave length can be computed as follows:

$$T = 16.5 \text{ seconds}$$

$$H = 99 \text{ feet}$$

$$L = 5.12\,T^2 = 1395 \text{ feet}$$

$$L/2 \doteq 700 \text{ feet}$$

A further simplification will then be made by assuming the wave form to be composed of triangles, as illustrated in Figure 6-1.

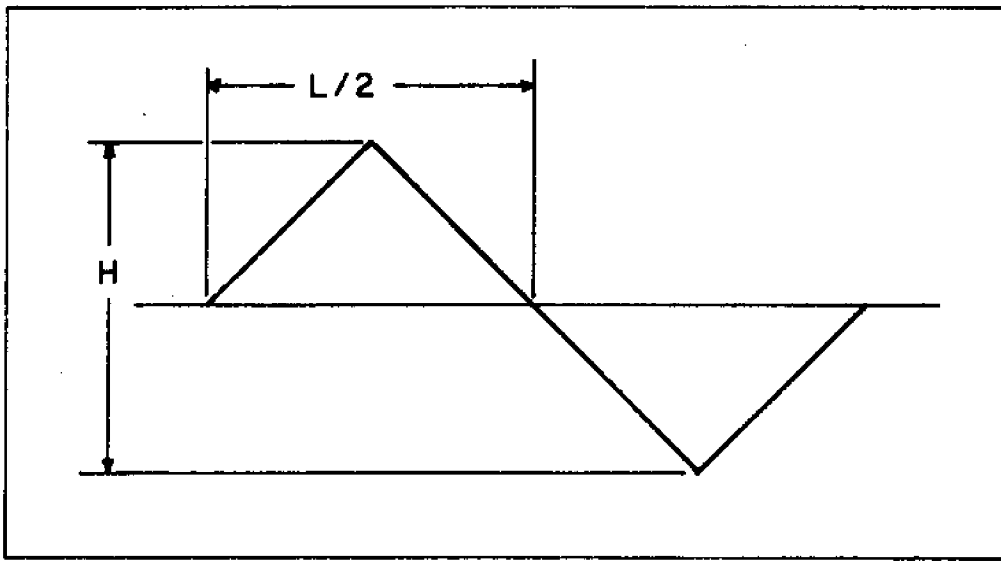


Figure 6-1. Simplified wave form

This assumption, while not accurate, will serve to give a reasonable estimate of the average change in water level. For greater accuracy, one should use solitary wave theory and compute the average change in water level. However, this level of accuracy was not considered necessary in this case. Since the length of the super structure of the nuclear power plant is 450 feet, while L/2 is 700 feet, as the design wave trough passes the structure an average change in water level of -33.9 feet will occur.

6.1.4 Total Level Change

Assuming the worst possible conditions created by the simultaneous occurrence of low tides, a tsunami and the design storm wave, a loading equivalent to 38.05 feet of water will occur.

Low Tide	3.15 feet
Tsunami	1.0 feet
Storm Wave	33.9 feet
	38.05 feet

In section 4.3.1, a displacement of 2,260 tons of water per foot of depth for the superstructure was calculated. Thus, under the worst possible conditions, the total weight to be supported by the foundation will be

$$35{,}000 + (38.05)\ (2{,}260) = 121{,}000 \text{ tons}$$

Here, an interesting problem develops. If we go the other direction and assume a high tide with 99 foot storm waves and a 1 foot tsunami, we find the rise in water level to be

Tide	12.6 feet
Storm	33.9 feet
Tsunami	1.0 feet
	47.5 feet

This 47.5 foot rise results in a decrease in weight of

$$W = (2{,}260)\ (47{,}5) = 107{,}200 \text{ tons}$$

Since the original weight assuming still water was only 35,000 tons negative, we now have a positive displacement of 72,200 tons.

This can be circumvented by either moving the structure to shallower water, adding about 70,000 tons of ballast or by securing the structure to the bottom with a mooring or anchoring scheme. Of the latter, one possibility might be rock bolts and cables. This would prevent the structure from lifting off the foundation as the design storm wave passed by, yet would not put increased loads on the foundation during normal weather conditions.

6.2 PILE FOUNDATIONS

In using piles as a foundation, several parameters must be considered. It must be known what types of soils and rocks are present, their bearing capacities or ability to provide support under varying load conditions, the sand covering and movement of sand, if any, the effect of corrosion on pile strength, etc.

For this particular foundation, the strength of each pile and the number of piles needed will be determined. It will be assumed a bedrock base exists, covered with a thin layer of sand (typical of the proposed location).

A hollow pretensioned concrete pile combined with a steel H-beam would be used. The pile is octagon shaped, 2 feet in diameter, with an H-beam in the center. This pile design was chosen because it provides high strength and good resistance to the corrosive affects of sea water.

The strength of the piles will now be determined. If a 6,000 psi concrete pile is prestressed to an effective prestress of 700 psi, the ultimate strength is computed as follows:

$$N' = (0.85 \times f_c - 0.60 \times f_s)\ A_c$$

$$= (0.85 \times 6000 - 0.61 \times 700)\ A_c$$

$$= 4680\ A_c$$

where N' is the ultimate strength of pile, f_c is the strength of concrete, and A_c is the cross-sectional area of the concrete pile in square inches.

The amount of prestress remaining in the tendons is approximately 60 percent of the effective prestress (f_s).

$$N = N'/4$$

Using a safety factor of 4, the design load (N) is one fourth of N', and

$$1170\ A_c = 1170 \times (24)^2 \times 3.14/4$$

$$N = 529{,}000 \text{ lb}$$

$$N = 264 \text{ tons}$$

Thus, for this 2 foot diameter pile, the design load is 264 tons.

There will be a net maximum weight of 121,000 tons after the entire structure is settled to the sea floor. The allowable design load is 264 ton/pile. The number of piles:

$$n = 121{,}000/264$$

$$= 458 \text{ piles}$$

The dimensions of the designed barge are 450 ft by 600 ft. Assuming a matrix arrangement:

$$3x \cdot 4x = 458$$

$$12\ x^2 = 458$$

$$x^2 = 38.2$$

$$x = 6.18$$

$$3x = 18.6$$

Piles will be arranged such that there will be 18.6 piles on one side and 24.8 piles on the other; or, 19 on one side and 25 on the other, thus allowing an extra margin of safety. Figure 6-2 illustrates the concept of a pile supported structure.

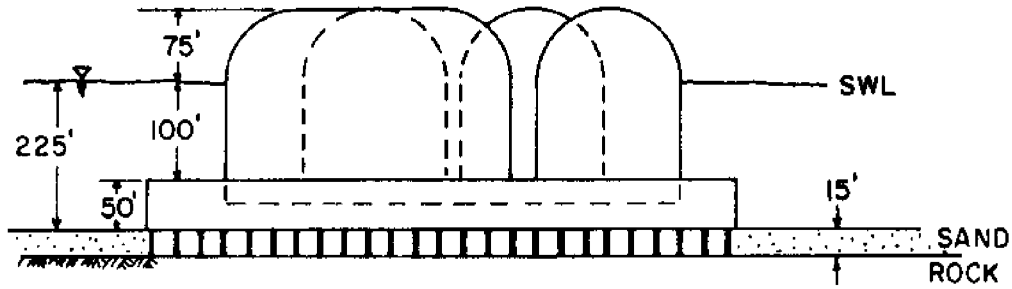
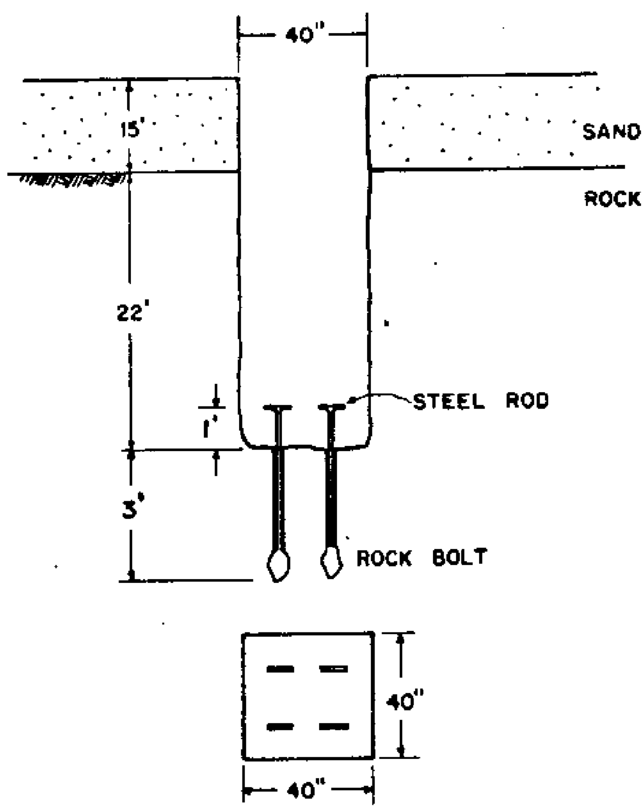


Figure 6-2. Pile supported nuclear plant.

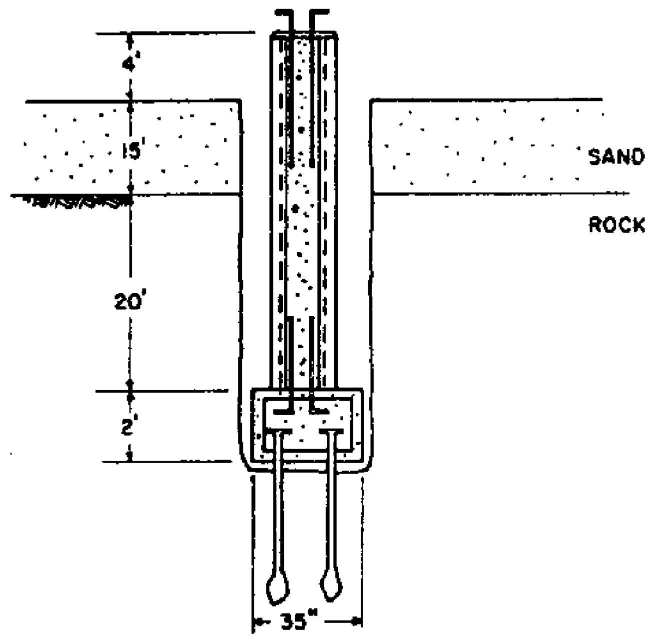
6.2.1 Site Preparation and Placement of Piles

The following series of sketches illustrate the steps in construction of the proposed pile foundation.



Step 1.

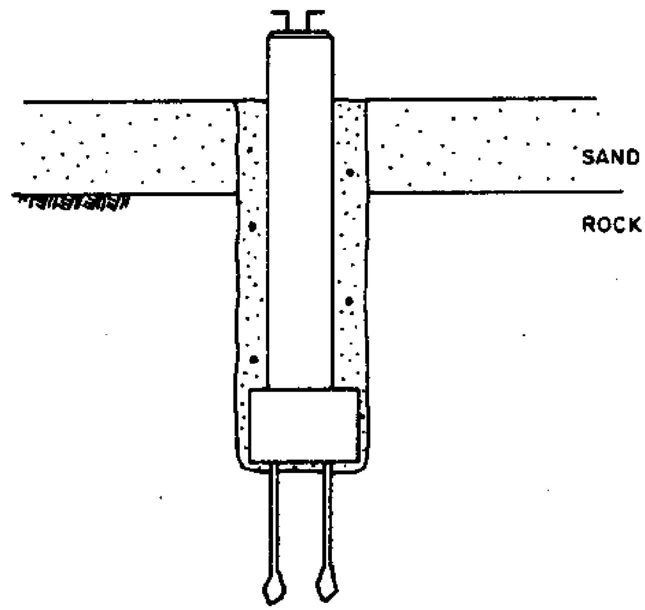
Step 1: Using techniques developed in marine construction, holes would be drilled at the desired positions. Then a rock tie would be secured to the bottom of the hole.



Step 2.

Step 2: The pre-cast pre-tensioned pile would be set into the excavation. Then the bottom of the hole would be filled with concrete to form the footing of the pile.

This could be accomplished by either pumping the concrete in, or devising a method that allows it to flow in. In pouring concrete under water, care must be taken to insure that the majority of the concrete is not exposed to the water; otherwise, the cememt may be leached from the concrete, and the structural strength would be inadequate.



Step 3.

Step 3. The remainder of the hole may be filled with grout.

It might be necessary to remove the sand before drilling the holes and inserting the piles. If this were thecase, an air lift is suggested as the fastest and most economical means of removing such a large volume of sand. Dredging at this depth with confidence over an extended time period may be very difficult.

6.3 FOUNDATION ALTERNATIVES

For areas where the use of piles may not be feasible, or if the expense of placing them is too great, other means of securing the nuclear power facility could be investigated. Following are a few of the possibilities.

6.3.1 Taut-Moored Facility

The possibility of leaving the nuclear power facility partially submerged, a taut-moored position, should be examined. This concept was proposed by Bascomb as a means of supporting a large floating bridge.[58] To gain some idea of the magnitude of the forces involved, the mooring system would have to withstand forces equivalent to pulling the entire structure 85.5 feet under water from the floating position. This would be

$$(64 \ \tfrac{\text{lbs}}{\text{ft}^3}) \ 85.5 \ (450') \ (600') = 1.48 \times 10^9 \text{ lbs} \cong 7.4 \times 10^5 \text{ tons}$$

This is due to the superposition of tides, tsunamis, and storm waves, as discussed earlier. The structure could either be moored with most of it out of the water, or almost totally submerged. In the first case, the facility could be moored in relatively shallow water (approximately 60 ft in depth). However, this advantage would be offset by the greater wind and wave loadings experienced by the structure. In the second case, one could virtually select any depth of water desired, up to the height of the structure, depending upon the particular physiography and geology of the area and how much of the facility it is desired to leave above water. To fasten the moorings to the ocean floor would probably involve using rock bolts or a similar system. It is not inconceivable that a means of adjusting the buoyancy of the structure to compensate for water level fluctuations could be devised, especially tidal variations. This alone would reduce the strains on the mooring system by a factor of one third.

6.3.2 Concrete Foundation

Instead of mounting the nuclear power structure on piles, another possibility would be to set it on concrete slabs. Even though this would involve using more concrete than the pile design, it might be less expensive and more satisfactory in the long run due to simplicity of construction. Techniques for the pouring of large slabs of concrete underwater have been developed fairly extensively in the field of port construction since the World War II.

6.3.3 Island Concept

The large forces encountered in the taut-mooring concept could be eliminated if the entire facility were placed inside a basin. This idea is not as improbable as it might seem, since there is a similar type structure now under construction off the coast of Norway in the Ekofisk oil field[60]. This structure is being constructed for the purpose of storing crude oil and utilizes a dual wall concept, with the outer wall being perforated to act as a breakwater. The size of the structure is 300' x 300', and is to be placed in 195' of water. This would also provide a measure of protection in the event a large ship collided with the structure.

7. Onshore Facilities

During and after construction of the nuclear power facility, it would be necessary to transport men and supplies to the site. Heavy refueling equipment could be loaded aboard barges and towed to the site from major ports along the coast; however, the problem of moving personnel and light supplies to the site would best be solved by the construction of a boat basin adjacent to the site. This would provide economical, safe transportation in a variety of weather conditions, and would continue to be of value to the area long after construction of the nuclear power facility was complete. The boat basin could provide moorage for fire boats and Coast Guard Vessels, and serve as a staging area for the supplies needed to support construction and operation of the plant.

7.1 BOAT BASIN DESIGN

Figure 7-1 illustrates a conceptual design of such a boat basin. Facilities would include:

a) loading crane - to facilitate loading of supplies and light equipment.

b) storage sheds - for protection of equipment and supplies from the weather.

c) living quarters - for guard personnel and other personnel who will live at the shore facility.

d) heliport - for rapid transportation in case of emergency.

e) barge and tug facilities - a small to medium-sized work barge and tugboats to support construction and operation of the plant.

Before construction of any such facility, a model study of breakwater positioning should be made to determine wave patterns about the breakwaters.

Data on littoral transport of sand in this area is not available; however, with Heceta Head to the north and Cape Perpetua to the south acting as barriers, this section of coast should be in equilibrium. Thus, while erosion and accretion due to the breakwaters may occur on a seasonal basis, interruption of the littoral transport should have minimal long term effects.

7.2 POWER TRANSMISSION TERMINAL

The shore facility would also include a terminal that would interface the submerged power cable with the power grid. This terminal is discussed in Section 4.

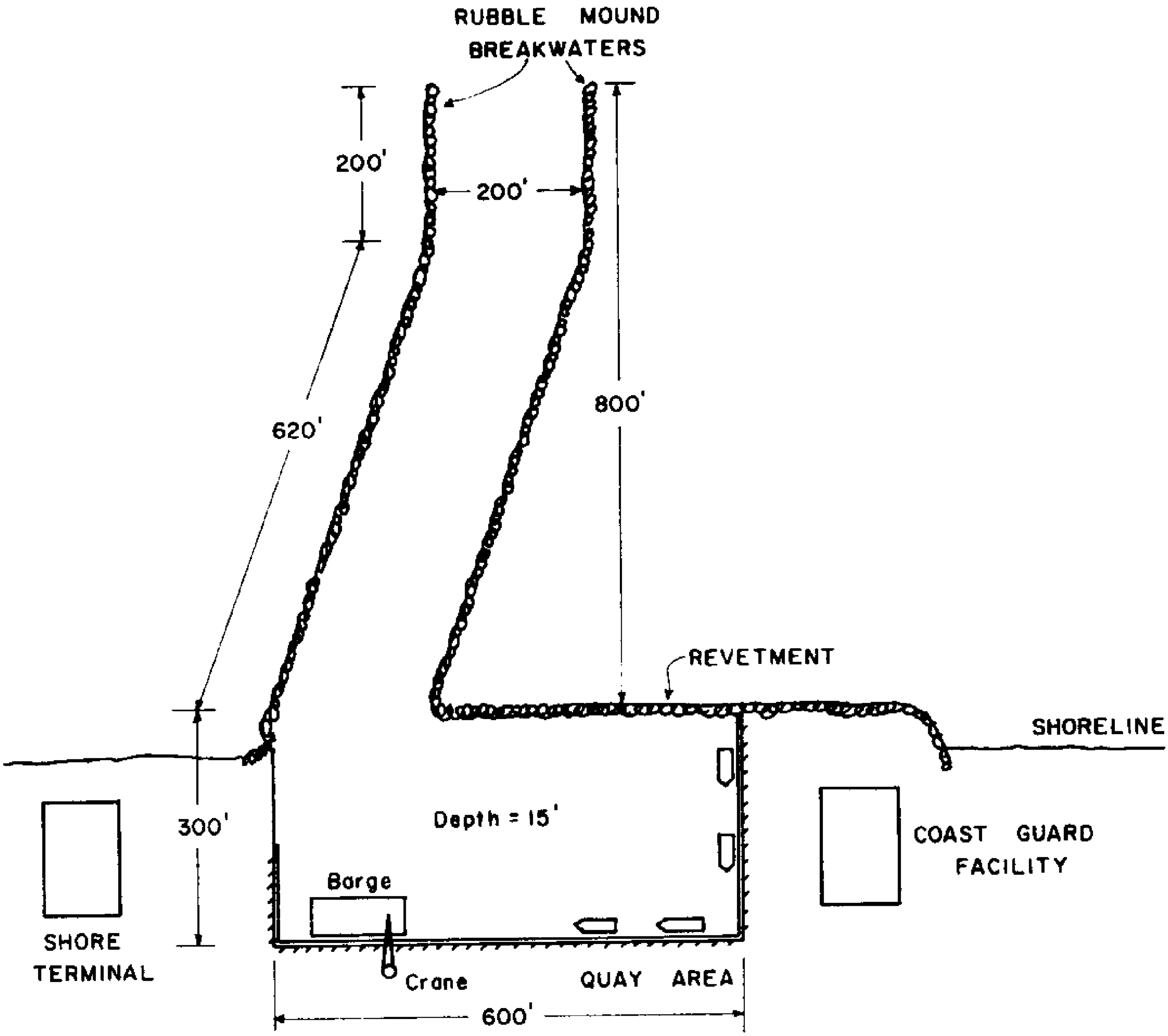


Figure 7-1. Conceptual sketch of boat basin.

8. Summary

The design presented in this report is a first attempt to assess the problems involved in siting a nuclear power plant offshore. Some of the expected problems have been given careful consideration, but this is by no means an all inclusive report. Our stated objectives were to determine the feasibility, practicality and desirability of an offshore nuclear power plant as well as to reasonably estimate the major design parameters. Offshore siting was found to be technically feasible and probably economically attractive.

8.1 CONCLUSIONS

In view of the fact that available land near a source of cooling water is becoming increasingly rare, it is felt that the placement of power generating facilities offshore will soon be the only acceptable alternative. It is felt this conclusion is reasonable in light of the fact that 18 to 20 nuclear power plants will have to be constructed to meet the predicted demands of the 1990's.

Socio-legal requirements have caused expensive delays and cancellations of inland nuclear generating facilities. The proposed offshore plant is felt to be more socially acceptable than onshore plants due to the minimal use of precious sea shore and the reduced potential for exposure of the near shore population to the genetic and somatic effects of long term, low level radiation. An offshore facility would allow the reactor to be physically removed from the population centers, and in addition would take advantage of the shielding capacity of sea water. Thermal impacts associated with land based plants can be minimized by offshore siting. It is felt that once through cooling by sea water is feasible, practical and acceptable from an ecological viewpoint.

The application of mass production techniques presently utilized in our Nation's shipyards has tremendous appeal. A skilled labor force could be maintained, eliminating disruptive moves from one site to another. Quality control on work would be improved with resultant savings if identical power plants were built at one location and transported by sea to their individual sites. Plants could be sold to other nations with minimum on-site construction. Mass production would allow such plants to be economically competitive with conventional land based designs.

Living conditions aboard an offshore semi-submerged plant do not appear to present any insurmountable problems. Life support systems aboard nuclear submarines operate routinely under conditions far more extreme than those to be expected on the proposed plant. Work schedules similar to those employed on offshore oil towers could be employed to avoid morale problems and high turnover rates among the crew members.

8.2 RECOMMENDATIONS

The suggested design characteristics of the proposed plant represent only one of many possible offshore alternatives for meeting future power requirements. Other techniques must be evaluated before an optimum combination of parameters can be achieved. In addition, there are many projects that are compatible with the operation of offshore thermal power plants that should be investigated, such as production of fresh water by desalinization, aquaculture, in-situ gathering and analysis of oceanographic and meteorological data, etc.

It is recommended that the following areas receive priority in assessing the future of offshore nuclear power plants:

1. Foundation design and/or anchoring techniques.

2. Economic considerations, including design analysis and aquaculture potentials.

3. Collision avoidance.

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