CHEVRON USA, INC.
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NKK AMERICA, INC.

A STUDY ON THE FEASIBILITY OF PRODUCTION, STORAGE AND LOADING SYSTEMS IN THE NORTH ALEUTIAN BASIN (OCS LEASE SALE 92)

SEPTEMBER 1985

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284/BWA

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October 7, 1985

To: North Aleutian (Lease Sale 92) Study Participants
(See Distribution List Attached)

Re: North Aleutian Basin (OCS Lease Sale 92)
Final Report

We are pleased to enclose the final documentation for this project. Included are two copies of:

- o The Final Report
- o The Executive Summary
- o Appendix A - Cost Data

The appendix contains a more detailed breakdown of the costs and includes original, unedited calculation sheets.

We enjoyed working on this project and should you have any questions on this report or any other matter concerning this project, please contact the undersigned.

Very truly yours,
BRIAN WATT ASSOCIATES, INC.

Jal Birdy

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Vice President

Enclosures

284/JNB/sll

TABLE OF CONTENTS

ITEM		PAGE

LIST OF TABLES

LIST OF FIGURES

LIST OF DRAWINGS

1.0 INTRODUCTION

This study was aimed at assessing the feasibility and costs of alternative production, storage and loading systems for application in the OCS 92 lease sale area in the North Aleutian Basin. The study was conducted by Brian Watt Associates, Inc. (BWA) of Houston, Texas, who was the main contractor. Fabrication, installation, costs and schedules for the steel jackets and hybrid structures was evaluated by M & R Enterprises of New Orleans, Louisiana, who acted as sub-contractor to BWA. The study was carried out for the following participants:

Chevron, U.S.A., Inc.
Conoco, Inc.
Minerals Management Service
Mobil Exploration and Production Services, Inc.
NKK America, Inc.

The lease sale area is shown in Figure 1.0.1. It consists of approximately 9,000 square miles of territory, with water depths ranging from 50 to 300 ft. The majority lies between the 200 and 300 ft contour lines. The area lies between the 55th and 57th parallels and as such no ice growth occurs in the region. The area is clear of ice for much of the year, but in the months between January and April the region is susceptible to relatively fast moving, rafted ice and first year ridge fragments drifting through and interacting with fixed or floating structures.

In addition to ice, the area is one of the most seismically active regions in Continental North America. The North Amak Fault lies close to the region and contributes to the severe seismic criteria mandated in this area. Earthquakes with magnitude 7 on the Richter scale have been experienced in the surrounding area in the past 30 years and there is historical evidence that earthquakes in excess of magnitude 8 on the Richter scale have occurred.

The lease sale area is prone to relatively severe wave conditions of magnitude similar to usual design criteria for the Gulf of Mexico.

With due regard to the environmental characteristics of the lease sale region a number of different concepts were assessed for their feasibility and costs. Among these were:

1) A piled steel jacket with remote storage and loading. This concept, shown in Figure 1.0.2, is typical of the many thousands of piled steel jackets used for production and drilling around the world.

2) A steel jacket with integral storage. This concept, shown in Figure 1.0.3, combines a concrete base used to provide storage and a steel tower used to support the deck. Loading and shipment of oil would take place from a remote loading buoy.

3) A concrete gravity platform with integral storage. This concept, shown in Figure 1.0.4 is typical of similar platforms used in the North Sea for production and storage. Loading and shipment of oil would be as in Concept 2.

4) A number of different tanker based floating production, storage, and loading system combinations were examined. A typical case is shown in Figure 1.0.5. Included were subsea templates and marine risers.

In addition to the basic concepts, the feasibility and costs of required ancilliary items such as pipelines were established. In all cases, base case scenarios were established and their sensitivity to various parameters, such as the environmental criteria, was determined. Particular emphasis was placed on the fixed platforms and especially the piled steel jacket concept. The floating concepts received a more general and less analytical review. Seismic loads were the principal environmental events studied for all fixed cases.

1.1 Study Scope and Objectives

The objectives of this study were as follows:

1) To assess the feasibility of alternative concepts for production platforms, storage and loading systems, and pipelines for use in the North Aleutian lease sale offshore Alaska, and,

2) To perform an economic trade-off between the alternative concepts, with particular reference to capital cost, schedule, cash flow and manpower requirements, and,

3) To study the sensitivity of the cost and feasibility of the concepts to changes in significant environmental parameters.

The method of approach used in this study was to perform an initial feasibility analysis which concentrated only on those parameters likely to significantly affect technical feasibility, cost and schedule. The intention was to develop the concepts in sufficient detail to establish that they were feasible and to provide major member sizes and dimensions suitable for the determination of required material weights, costs and schedules. No sophisticated analysis of factors affecting cost and feasibility in a secondary way was performed.

While a general approach on a relatively large number of concepts, rather than a detailed analysis of a single concept, was used in the study, the methodology used was state-of-the-art. Seismic analysis was performed using both response spectrum and time history methods, including the generation of artificial earthquakes, and both ice and wave load analysis were based on recognized design procedures.

In developing the concepts, the approach taken was to establish a series of base case scenarios where the following variables were fixed:

- deck weight
- water depth

- seismic criteria
- ice criteria
- wind and wave criteria
- installation methods

For each of these parameters, the sensitivity of the concept to changes within realistic ranges was established. The specific base case values and sensitivity ranges are discussed in the various concepts.

The approach and methodology used are consistent with realizing the objectives of this study. The concepts in no sense represent a bid level design, but they have been established using realistic and sophisticated engineering analysis and represent a realistic basis for establishing material, cost, and schedule requirements for various drilling and production systems in the North Aleutian Basin.

In the design of all the fixed structure concepts the deck design was not included. Suitable provision was made for the deck in terms of weight, cost and schedule by comparison with existing decks on similar structures.

Attention in this study was focused on the capital costs of the installed hardware only. No costs are included for well drilling, supply, logistics, operation, maintenance, etc. It was assumed that participants would carry out financial analyses to meet individual needs based on the supplied information.

1.2 Report Organization

This report is grouped loosely into four major divisions. In the first division, Chapters 2 through 4, describes the environmental criteria developed for the North Aleutian Basin and a general description of the proposed concepts is given in Chapter 3. The methodology used in developing costs and schedules is given in Chapter 4.

In Chapter 5 through 8, the three fixed structure concepts are described in detail. For each concept the method of approach and analysis methods are discussed together with a summary of the major analysis results. The costs and schedule for the construction of the specific concept is discussed and the sensitivity to changes in various environmental operational parameters is established. Conclusions and recommendations are given for each concept. Each of the three fixed structure concepts is discussed in a self contained format which allows that concept to be considered separately. In Chapter 8, the implications of potential liquefaction, which affects all three fixed platforms, are discussed.

The floating production and storage concepts are discussed in a similar way in Chapters 9 and 10. The emphasis is on a concise description of the concept, its cost, schedule, and sensitivity. In addition, factors of particular importance to floating systems, such as weather related downtime were evaluated.

In Chapter 11, pipelines which are common to all field development scenarios are discussed. Pipeline size selection and cost are discussed. A series of realistic field development scenarios using the various concepts were developed in Chapter 12. The capital costs implied in the development plan were evaluated. General conclusions and recommendations for the study as a whole are presented in Chapter 13.

A summary of the estimated costs and assumptions made in the cost estimates is given in this report. A more detailed breakdown of the costing procedure is given in Appendix A to this report under separate cover. The reader is cautioned against directly comparing fixed platform and floating system costs. The initial capital costs for fixed platforms include drilling rig and equipment capital costs. The floating system does not. Other significant differences are pointed out in the text.

2.0 DESIGN CRITERIA FOR THE NORTH ALEUTIAN LEASE SALE AREA

The North Aleutian Basin lease sale area, OCS 92, lies off the west coast of Alaska, to the North of the Unimak Islands and the Alaskan Penninsula and to the east of St. George Basin and the Pribilof Islands. It is approximately 600 miles southwest of Anchorage and lies offshore in a largely uninhabited and undeveloped region with water depths ranging from 50 to 300 ft in the lease sale area. The area is shown in Figure 2.0.1.

Because of the active seismic nature of the area and the presence of faults, the major design level environmental loading on fixed platforms will be derived from the strength level earthquake. In addition, because of the presence of drifting ice in the region the fixed and floating systems must be analyzed under design ice level forces. Wind and wave conditions are required for fixed structure design and for assessing the operational and survival capabilities of floating systems.

The various environmental and operational criteria, required for this study, are described in the following sections.

2.1 Atmospheric Conditions, Temperature and Ice Accretion Criteria

The lease sale area lies on the continental shelf, southwest of Bristol Bay and to the north of the Aleutian Islands in a region that is dominated by maritime influences as shown in Figure 2.1.1. The area is characterized by relatively low temperature variations and a moderate climate for this region, as shown in Figure 2.1.2. The principal data source used for determining environmental data was Reference 1. Additional data was obtained from References 2 through 5.

The mean air temperature during the winter months is moderate and the limited number of freezing degree days ensures no growth of sea ice in the region. The average annual maximum temperature for the region is 5.0°C (41°F) and the average annual minimum temperature is -0.8°C (31°F), indicating a relatively cool summer and warm winter climate.

The average annual precipitation in the lease sale area is 22.6 inches, with percipitation of one form or another occurring almost 25% of the year as shown in Table 2.1.1.

A condition of significant interest for loading and transportation of crude, is high annual occurrence of reduced visibility in the lease sale region. Because of the closeness between the air temperature and the seawater temperature over much of the year, fog is a common occurrence in the lease sale area. Percentages of occurrence of reduced visibility, defined as visibility less than 5 miles, obtained from various locations around the lease sale area are given in Figure 2.1.3. Visibility is reduced by fog approximately half the time in summer. In the winter months the occurrence of snowfall and blowing snow offshore will further reduce visibility. The estimated average annual frequency of occurrence of reduced visibility due to all sources is 28.8%, but this number varies significantly over the lease sale area, increasing from east to west. The issue of fog and reduced visibility is further discussed in Chapter 10 where weather downtime associated with offshore loading operations is discussed.

While no sea ice growth occurs in the region, in the computation of deck weight for fixed platforms and in the computation of heating requirements needed on floating production systems the accretion of ice on exposed surfaces must be included. Ice accretion can be extremely severe in this region and data available from References 6 through 8 indicates that accretion rates can range from 1.25 to an extreme value 3.75 inches per 3 hour time period.

2.2 Bathymetry, Wind and Waves

In addition to References 1 through 5, the major references consulted for wind, wave and current data were References 9 through 14. The lease sale area shown in Figure 2.2.1 covers an area of 9,000 square miles. The bathymetry is relatively flat. Water depths range from a localized low value of 50 ft in the extreme northeastern region to a maximum of 300 ft in the northwestern lease sale area. The majority of the lease sale area has a water depth of between 200 and 300 ft.

Offshore wind conditions are extremely variable in the area. The predominant wind direction in summer is from the south while in winter the dominant wind direction is from the North. The annual mean wind statistics are given in Table 2.2.1. Light summer wind conditions are indicated with maximum wind conditions likely in the early winter months.

The design wind speed conditions as a function of return period are given in Table 2.2.2. For a 100 year return period the design wind speed is 106 knots. The one minute average wind speeds are given in this Table. Wind speeds referred to other time periods can be developed according to the rules specified in Reference 19.

The wave conditions over the area are characteristic of a seastate dominated by short duration storms, a limited fetch and the relatively shallow depth. The Aleutian Islands provide shelter for the region and the variable wind climate produces mainly short period, choppy seas. In summer the seastate is dominated by westerly wave conditions. Only 1% of the waves in an average summer exceed 12 ft and 75% of the wave periods are below 6 seconds. In winter the wave climate shifts to the North and Northwest increasing both in magnitude and period. Typically 99% of the waves are less than 20 ft in winter. The summer (June), winter (November) and long term wave climate is given in Figure 2.2.2.

The design wave conditions for the fixed platforms are summarized in Table 2.2.3 for the maximum (300 ft) water depth and presented in Figure 2.2.3 as a function of water depth over the lease sale area. The maximum design wave height for fixed structures is 71 ft, with the peak spectral period of the associated JONSWAP spectrum being 14 seconds.

The current regime in the lease sale area is influenced primarily by the Alaska Current and the topography of the Unimak Pass and Isanotski Strait. The area tidal currents are driven by the counterclockwise gyre in Bristol Bay and the prevailing winds. At the various passes, tidal currents can reach 4 knots. The design level extreme surface currents are given in Table 2.2.4.

The mean tidal range is expected to be 6 ft offshore in the North Aleutian region, while nearshore in various bays and inlets tides in the 10 to 14 ft range occur. In addition the expected storm surge for use with maximum wave conditions is 2 ft. In the nearshore regions storm surges in the 4 to 6 ft range are possible.

A summary of the wave, wind, current and tidal climates used in the design is given in Table 2.2.5.

2.3 Seismic Design Criteria

The southern coast of Alaska and particularly the North Aleutian Basin is an active and severe seismic region. The northern regions of the lease sale area are influenced by the North Amak Fault Zone which probably penetrates directly into the lease sale area. In the past 30 years earthquakes centered in the region have occurred with a Richter magnitude of at least 7 and historical evidence indicates that Richter magnitudes exceeding 8 have occurred in the lease sale area. A summary of recorded seismic activity in the general area, extracted from the OASES study, Reference 16, is given in Figure 2.3.1.

In developing seismic design criteria for fixed and floating structures, various descriptions of an earthquake are used in practice. The most comprehensive description of an earthquake is by means of a time history such as that shown in Figure 2.3.2. This provides complete information on the ground motion. In the North Aleutian Basin, all earthquakes have been measured at land based stations, remote from the region. Hence, no actual ground motion traces are available for specific sites in the lease sale area.

Simpler measures of seismic activity are based on characteristic values of the earthquake, such as peak ground acceleration, maximum pseudo velocity, etc. These parameters in themselves are not sufficient to describe an earthquake. They are generally coupled with a measure of the frequency content of the earthquake in the form of a response spectrum. A typical response spectrum is shown in Figure 2.3.3.

The approach taken in this study has been to characterize the seismic behavior by means of the response spectrum method. This is consistent with the non-availability of site specific data and the requirement to provide a realistic and safe envelope for design purposes. In situations where time histories were required for analysis, the approach has been to generate a number of artificial earthquakes all of which provided the target design spectrum. The artificial records were generated using the program SIMQKE, Reference 15.

The most comprehensive public domain study of seismic criteria in the region is the OASES study, Reference 16. Contours of maximum acceleration with a nominal return period of 100 years are provided. Peak ground accelerations of between 0.15 g and 0.35 g are indicated, as shown in Figure 2.3.4. In the design of fixed platforms, the usual target return period is 200 years. It should be stressed that, especially in seismic analysis, return periods should not be viewed in an absolute sense but rather as a relative measure of seismic severity.

The design response spectrum, selected in consultation with the study participants is shown in Figure 2.3.4. The peak ground acceleration is approximately 0.35 g. For comparison purposes the design response spectrum is compared with the API, Zone 3 response spectrum, Reference 20, specified for the general lease sale area in Figure 2.3.5. The design spectrum has a spectral acceleration which is typically about 50% higher than the API spectrum over the frequency range of interest in this study (approximately 0 to 2 second periods). In effect the design spectrum lies close to the maximum API Zone 5 criteria.

In applying the spectrum it has been assumed that the earthquake is measured at the mudline, i.e., all local site dependent soil amplification effects have been included in the spectral shape. To establish the sensitivity of the structural concepts to the uncertainties in the definition of seismic criteria the API Zone 3 criteria were also applied to a limited number of cases. The directional rules for combining earthquakes specified by API in Reference

20 were used in the structural design, i.e., components of 1 and 0.67 in the horizontal direction and a 0.5 component vertically were superimposed.

2.4 Sea Ice Criteria

The small number of freezing degree days eliminates any potential for sea ice growth in the region. The region is characterized by drifing ice floes during a season which lasts from early January to late April. Coverage on the average is approximately 50% in the most northerly areas of the lease sale region. Based on the average of 17 years data, Reference 17, for the northern edge of the area, there is no ice 25% of the season and coverage is less than 75% for three quarters of the ice season. However, it should be noted that 100% coverage does occur. The occurrence of ice decreases rapidly towards the south. The presence of ice on a half month basis is shown schematically in Figures 2.4.1 and 2.4.2. Ice incursions below the 56th parallel, i.e., for the southern half of the lease sale area, are extremely rare.

In addition to ice sheets, first year ice floes, having predominantly rounded edges due to ice/ice collisions and ice/water interaction do occur. These floes are generally less than one quarter mile in diameter. Coverage of the area with floes is generally less than 10%, although up to 40% coverage with floes has been observed.

The principal design requirements for fixed and floating structures are:

1) Global forces on the complete system

2) A pressure area criterion for the local design of structural components and members.

For the purposes of meeting these requirements, data is required for:

- Ice thickness
- Ice velocity

- Ice strength

Based on the number of freezing degree days, the anticipated ice growth in regions surrounding the lease sale area is given in Table 2.4.1. The proposed sheet ice design thickness for use in the lease sale area is 24 inches. This is compared in Table 2.4.2 with sheet ice thicknesses previously used in the design of existing structures in other areas. The design thickness for rafted ice is assumed to be a double sheet thickness or 48 inch.

In addition to the sheet ice loads, a first year ridge ice thickness of 3 times the sheet ice thickness or 72 inches has been used, for reasons to be explained in the following.

The standard procedures for computing ice loads on fixed structures are defined in API, Bulletin 2N, Reference 27. For a floe or sheet impinging on a circular member

$$F = I * f * C * (D * t) \tag{2.4.1}$$

where

I	is an indentation factor
f	a contact factor
C	the unconfined compressive strength
D,t	the member diameter and thickness

The first three factors I * f * C represent the ice pressure. Rather than trying to estimate the parameters individually, the approach taken here has been to estimate the ice pressure by inference from the measured data presented by Blenkarn, Reference 17. From measurements made in 1964 through 1968 in Cook Inlet, average crushing pressures of 20 to 160 psi were measured on ice thicknesses of 5 to 45 inches at effective strain rates of 0.14 to 0.24/sec. Assuming the form of the unconfined compression strength curve shown in Figure 2.4.3 and for an estimated strain range of 0.005 to 0.24/sec for the structural member diameters used in the proposed concepts, a similar pressure can be anticipated in the North Aleutian Basin. An effective crushing pressure of 160 psi has been assumed for the computation of ice loads. Because the test results in Reference 17 produced ridge loads of three times the magnitude derived from the sheet ice, a ridge thickness of 6 ft (3 thicknesses) has been used for ridge load computation. The maximum ice load on a large area of a structure is thus,

$$3 \times 2 \text{ ft} \times 160 \text{ psi}$$
$$= 138 \text{ kips/ft}$$

The design ice loads proposed here are compared with existing Cook Inlet structures in Table 2.4.3.

The pressure area curve used for the local design of the members is given in Figure 2.4.4. The maximum pressure of 400 psi reducing to 160 psi for areas greater than 100 ft^2 is based on the relatively high temperature of the first year ice which has been shown to produce effective pressures well below those proposed for the cold, multi-year Beaufort Sea ice.

2.5 Geotechnical Criteria

In the present study only limited application was intended for the geotechnical criteria. While penetration lengths for piles in the jacket concepts and global geotechnical stability for the gravity systems was performed, no dynamic analysis of the pile/soil system under earthquake loading was undertaken. Two soil profiles shown in Figure 2.5.1 were used in this study. Profile A has a 60 ft layer of dense sand overlaying stiff clay with an undrained shear strength rising from 2 ksf at 60 ft to 5 ksf at a 500 ft depth. Profile B has a 20 ft layer of sand overlaying a 55 ft clay layer which in turn overlays dense silty sand.

2.6 Platform Function and Operational Criteria

Two base case production scenarios were specified for the study. The scenarios for 50,000 and 100,000 bpd production rates are defined in Tables 2.6.1 through 2.6.4. For fixed platforms, the most important parameter is the weight and eccentricity of the deck and its equipment. In the seismic environment deck load is significant. The estimated total deck load is 60,000 kips for a 100,000 bopd production rate. For the purposes of design, a base case deck load of 60,000 kips was assumed and in the sensitivity studies performed, the deck load was varied from 40,000 to 100,000 kips.

The anticipated deck layouts are given in Figures 2.6.1 through 2.6.4. A major concern in the concrete gravity structure of Figure 1.0.4 was to ensure sufficient clearance internally in the tower legs for the required number to conductors. Typical layouts of the conductor system in circular legs are shown in Figure 2.6.5.

2.7 Design Codes

The design codes used in this study are as follows:

Primary Codes

Steel Jacket & Piles	API RP 2A, Reference 20
Concrete Structures and Foundation	ACI 357, Reference 28
Floating Systems	ABS Rules, Reference 29

Secondary Codes

Applicable DnV Code, Reference 19 or 30

3.0 CONCEPT DESCRIPTIONS

The water depth range in the lease sale area is potentially suited to the application of a wide variety of concepts and field development plans. The major requirements in any chosen system are as follows:

1. A production capability. Two different production capabilities were examined in this study, including a conventional fixed platform deck, and a tanker based floating production system coupled with a subsea template system.

2. A storage capability. In this study two different storage capabilities were explicitly examined. In two of the fixed platform concepts discussed in Chapter 1, an integral storage capability was provided in the base caisson. In addition for fixed platforms without storage, a tanker based floating storage capability was assessed.

3. A transport capability. For all concepts considered, a shuttle tanker system was assumed to transport the produced crude to the U.S. West Coast. In addition, the feasibility and costs of pipelines between integral storage systems, and remote loading buoys was considered.

The fixed platforms considered included both steel, concrete, and hybrid steel-concrete systems. All floating production, storage, and transport systems were assumed to be tanker based. A description of the development of each system is given in the following sections.

3.1 Jacket Structures

The conventional steel jacket system shown in Figure 3.1.1 was considered for all water depths over the lease sale area. In developing the jacket outline, several considerations and initial assumptions were required. To represent the lease sale area and to assess the sensitivity of cost, schedule, and feasibility to water depth, two target values of 150 ft and 300 ft were selected for this parameter. These depths bound the majority of the lease sale area where steel jacket platforms would be installed.

The lease sale area is subjected to severe seismic criteria as discussed in Chapter 2. It was anticipated initially, and confirmed subsequently, that for single jackets carrying all the production equipment, the loadout weight of the jacket would far exceed lifting capacities routinely available (up to 3,000 tons) and hence, launch trusses were incorporated into all the single jacket concepts in both 150 and 300 ft water depths.

The placement of the deck on the jacket was considered to have no influence on the member layout allowed for the jacket. In the case where an integrated deck is considered, essentially fully constructed and hooked up prior to mating with the jacket, additional restrictions would be imposed on the layout of the jacket framing and/or the mating and completion offshore of the jacket/deck system. These restrictions were not explicitly considered in this study, either in the feasibility or cost of completing the deck/jacket system in this way. The methodology and costing used in the deck/jacket design and installation are consistent with lifting the deck into place in sections as is the case in typical Gulf of Mexico and North Sea situations. This approach was taken because the use of a floatover deck is severely restricted by several factors. The wave climate in exposed areas makes installation difficult. The presence of ice results in a protection system for the conductors which would interfere with the deck installation process. Of most concern, however, is the requirements imposed by seismic loading. Cross bracing through the waterline is essential and unless substantial and expensive offshore construction of these braces were undertaken, standard floatover procedures would be impossible.

Two options were available for the placement of conductors. The first option considered was to place the conductors in large diameter legs and to drill through the legs. The major advantage of this system was that the conductors were enclosed and protected from direct ice attack. This system has been employed particularly in Cook Inlet, where similar ice conditions exist. The procedure of drilling through the legs, however, also had several significant drawbacks. For production rates in the range of 50,000 bopd to 100,000 bopd, leg diameters in the 40 to 55 ft range were required. The use of these large diameter legs would place the piles, supporting the axial and shear loads in close proximity to the conductor system, leading to a reduced and more uncertain capacity for the piles because of interference and driving effects from the conductors. In addition, the close proximity of piles and conductors is undesireable in light of potential drilling accidents.

In conventional systems, the large diameter legs would be vertical. The severe seismic criteria ensure that the shears and flexural stresses on the piles at the mudline region will be severe. Under these circumstances, a battered pile system is highly desireable.

After a careful review of the potential advantages and disadvantages, the option of drilling through the legs was abandoned. For both the 150 ft and 300 ft water depths an eight leg jacket structure with one main pile per leg and an additional number of skirt piles, determined as required, to adequately support the platform was selected. The conductor system was positioned clear of the legs and protected by a cage system extending 15 ft above the mean water level and almost 100 ft below the mean water level depending on the water depth. This ensures that the conductors are adequately protected from unbroken ice, allows the use of double batter with a launch truss, and separates the piles and conductors.

The concepts selected for study in 150 and 300 ft water depths are shown in Figure 3.1.1. In each case it was assumed that all deck facilities and equipment were carried on a single platform. In addition to the options of using a single platform in all water depths, the use of multiple four leg platforms with launch trusses, each carrying a portion of the facilities weights was examined and is discussed in Chapter 5. The rationale for the selection and layout of these jackets was based largely on the same parameters as discussed in this section, with the reduced deck area and deck loads allowing a reduction to four legs. A typical elevation for a multiple platform in 150 ft water depth is shown in Figure 3.1.1.

3.2 Hybrid Steel/Concrete Structure

The hybrid structure shown in Figure 3.2.1 combined a concrete caisson used for integral oil storage, with a steel jacket used for support of the production and facilities decks. The jacket structure was designed based on similar considerations as those described for the pile structure in Section 3.1. The jacket legs have a double batter and have a steel cage system protecting the conductors. As for the piled jacket, two base case water depths were considered, specifically, 150 and 300 ft water depths.

The size and dimensions of the base caisson are controlled by a number of variables. A target storage quantity of approximately 9 days production was used, leading to a minimum storage requirement of 900,000 barrels for the 100,000 bopd production case and 450,000 barrels for the 50,000 bopd case. In addition, the base caisson was sized to provide sufficient volume for the ballast material required to resist sliding under wave, ice and seismic conditions and to allow a 20 ft water/emulsion layer to be positioned between the stored oil and the ballast material.

In developing the concept, it was assumed that the caisson would be constructed separately and towed to an inshore mating site. The steel jacket would be lifted on and welded in sections. The deck and associated modules would be placed in several lifts at the inshore site. It was also assumed that the jacket legs would be grouted directly into slots in the base caisson. The completed facility would be towed to location and installed on the seabed.

The concept was developed for both 300 ft and 150 ft water depths. An additional consideration in the lower water depth was to minimize the interaction of the caisson top and wave zone. For this reason, the height of the caisson in the 150 ft water depth was limited to 90 ft.

3.3 Concrete Gravity Platform

The concrete gravity platform was designed to the same criteria and base cases as the other fixed platforms described in the previous two sections. The platform outline is shown in Figure 3.3.1. The base caisson for oil storage was designed as discussed in the previous section for the hybrid structure.

The concrete tower sections were designed to meet the operational and environmental criteria. Two distinct tower leg types were used. Two large diameter drilling towers have the inside diameter controlled by the well spacing requirements. The minimum diameter for the 100,000 bopd production case is 56' 10". The remaining two towers are used for various utilities and have an outside diameter of 40 ft. The lower regions of these towers are tapered to provide the same base diameter as the drilling towers. This facilitates the intersection between the towers and the base caisson.

The tower wall thicknesses are tapered according to the requirements imposed by the applied bending moment. The walls have both nonprestressed steel and prestressing steel as required by the imposed stresses from seismic, ice and wave loading.

3.4 Floating Production Storage and Offloading (FPSO) and Floating Storage Offloading (FSO) Systems

A number of alternative tanker based floating production and storage systems were intially screened. The original candidate systems examined are shown in Figures 3.4.1 through 3.4.5. All the systems are connected to a total of eight subsea templates, each having eight slots, one which is reserved for gas injection purposes. The principal differences between the systems are:

1. Type of riser, i.e., flexible or rigid.

2. The mooring system, i.e., catenary or single anchor leg, catenary anchored tower or catenary anchored buoy.

In all cases, a 126,000 DWT tanker was used for floating production and/or storage analyses. A single water depth of 250 ft was used for the purposes of design.

To assist in screening the concepts, preliminary analyses were conducted on a number of key components. The seastate spectra with return periods of 100 and 1 year are shown in Figure 3.4.6. The response amplitude operators for the tanker are given in Figure 3.4.7. A summary of anticipated tanker motions is given in Table 3.4.1, for the survival condition in a 100 year storm. The estimated total forces on the tanker from various sources are given in Table 3.4.2. The steady design load for the mooring system, combined with an anticipated first order surge of 15 ft was used to design the mooring system. A symmetric, 8 leg system using 5 3/4" grade 3 chains and a chain length of 3,060 ft, was determined. A curve showing force against excursion in the individual moorings is shown in Figure 3.4.8.

It was assumed in the study that floating production and/or storage tankers would be ice strengthened. An allowance was made in the cost estimate for limited ice strengthening of the hulls. It was noted in Chapter 2 that over much of the region ice occurs very infrequently.

In the initial phase of the study, a total of 10 subsea templates were envisaged. The initial layout for the proposed flexible riser and rigid riser is shown in Figures 3.4.9 and 3.4.10. As the study progressed the number of templates was reduced to 7 producing templates and 1 for gas reinjection and the riser layout was modified to that discussed in Chapter 8.

The buoy sizes required with both rigid and flexible risers were estimated and the forces on the buoys computed in Tables 3.4.3 and 3.4.4. In the case where the buoy is restrained against vertical motion by the risers almost 350 tons of force is generated on the rigid risers.

The basic feasibility checks described here, indicate that all the concepts appear at least superficially to be practical. The actual selection process for the optimum or ideal concept is best performed by a detailed study and trade-off analysis of each type. Consistent with the scope of this study a subjective but realistic comparison study was performed. Each system was ranked in 6 broad categories as follows.

- o Technical Merit (40)
- o Impact on Captive Tanker (25)
- o Hull and Outfitting Cost (35)
- o Mooring and Riser Cost (35)
- o Operating Cost (30)
- o Reliability and Safety (35)

These six categories were each assigned the relative weight, shown in parenthesis, i.e., for example "Technical Merit" was ranked to have a relative weight of 40/25 = 1.6 times the weight of "Impact on Captive Tanker."

Each of these main categories was subdivided into a number of subcategories, which are listed in Tables 3.4.5 through 3.4.10. Each subcategory was assigned a basic score of 1 to 10, 10 being the best. In addition, a relative weight was assigned to each subcategory.

The scores and weights for each system by category are shown in Tables 3.4.5 through 3.4.10. An overall score for the total system is given in Table 3.4.11. The "best" system according to this method is a Turret Moored Tanker with a Flexible Riser. This system was judged to be the most reliable, have the lowest operating, mooring and riser costs and to have the most technical merit. These factors outweighed the increased impact of the system on the captive tanker and the resulting increased hull and outfitting costs associated with the turret mooring system.

For floating production and floating storage applications, the results of this ranking excercise indicated that the TMFR (Turret Moored Flexible Riser) scheme was optimum. This was the basis of the selection of this scheme for more detailed study and cost estimates.

3.5 Shuttle Tanker and Remote Loading Systems

All the concepts proposed in this study, both fixed and floating could require a shuttle tanker system to transport crude. In cases where FPSO or FSO systems are used, the shuttle tanker will be moored in tandem at the stern of the captive tanker. For the hybrid and concrete gravity systems a remote loading system will be required in conjuction with the shuttle tankers.

Three of the systems previously examined for the FPSO/FSO tanker system were also used in an investigation of shuttle tanker and remote loading systems. An additional concept was also examined for this requirement. The CAM system of Figure 3.5.1 has the advantage that when the remote loading operation is complete, the mooring and loading hardware will automatically be lowered out of the potential ice zone.

The shuttle tanker was assumed to have a displacement of 60,000 DWT. A preliminary investigation of the motions was made. The RAO's of the tanker are given in Figure 3.5.2, the extreme motions in the 1 year storm are given in Table 3.5.1 and the mooring forces in Figure 3.5.3.

A procedure identical to that used in the evaluation of captive tanker systems was used to select the shuttle tanker. The major categories are similar to those previously used:

- Technical Merit (46)
- Hull and Outfitting Cost (40)
- Mooring/Riser Cost (40)
- Operating Cost (34)
- Reliability and Safety (40)

The ranking study is summarized in Tables 3.5.2 through 3.5.7. The CAM system of Figure 3.5.1 was judged to be the optimum for the remote loading system. This system has signficant technical merit, performs well in ice, and has superior reliability and safety. The operating and hardware costs are competitive. As a result of this excercise the CAM system was selected as the optimum case for further study.

3.6 Pipelines

The use of pipelines to transport crude at the production rate was considered in conjunction with fixed structures. In the case of the concrete gravity and hybrid concepts a one mile length of pipeline was considered to transport crude to the remote loading buoy. In the case of the jacket system, a pipeline may be required to transport crude to a floating or buffer storage system. In addition the possibility of using a pipeline to shore for the jacket system was briefly investigated.

Pipelines were also used in conjunction with floating production systems in conjunction with the subsea templates. A total of 8 pipelines were required to connect the flexible risers to the individual templates.

3.7 Field Development Scenarios

A total of five field development scenarios were considered, for all water depths, and for all production rates defined in this study.

Scenario 1: A jacket structure with no integral or floating storage connected by a marine pipeline to shore. While not explicitly considered in this study it is expected that a landbased pipeline would link this system to a loading terminal.

Scenario 2: A jacket structure connected by a one mile pipeline to a Turret Moored Flexible Riser (TMFR) floating storage tanker with shuttle tankers to transport the crude.

Scenario 3: A hybrid structure connected by a one mile pipeline to a remote loading buoy with shuttle tankers for crude transportation.

Scenario 4: A concrete gravity structure connected by a one mile pipeline to a remote loading buoy with shuttle tankers for crude transport.

Scenario 5: An FPSO, with 8 subsea templates, connected by 8 x two miles of pipeline to flexible risers, with shuttle tankers for transport.

The floating production and/or storage assumes a TMFR tanker based system.

4.0 COST BASIS AND SCHEDULE DETERMINATION

A major objective of this study was to produce realistic estimates of the required construction and installation schedule, together with capital cost and cash flow requirements. Only capital costs were considered. No annual operating, maintainance, supply, drilling, crude transport, logistics, etc., costs were included in the cost estimates. The cost information was developed for the following specific cases.

- Steel Jacket Platforms
- Hybrid Platform
- Concrete Gravity Platform
- Floating Production Storage and Offloading System
- Remote Loading/Mooring System for Shuttle Tankers
- Seabed Pipelines
- Subsea Templates

In developing cost and schedule information, all fixed platforms were assumed to be constructed in Japan and towed to the North Aleutian Basin installation site. As an alternative, the cost sensitivity of constructing the platforms on the West Coast was examined. For all fixed platforms two production rates, 50,000 and 100,000 bopd were considered. In addition two water depths, 150 and 300 ft were assessed. For floating systems a single 250 ft water depth was analyzed.

In developing the cost data, the construction, transportation, and installation were divided into a number of key items. For a fixed structure the major items considered were:

- Material Procurement and Cost

- Jacket or Concrete Tower Fabrication, Transportation, and Installation.

- o Deck Fabrication, Transport, and Installation
- o Pile Fabrication, Transport, and Installation
- o Module Transport, Hookup, Installation, and Commissioning
- o Base Caisson Construction, Transport and Installation
- o Inshore and Offshore Mating Procedures, Schedules and Costs
- o Required pipelines, templates, and offloading systems

The various floating systems were costed for primary items including:

- o Tanker and Equipment Procurement and Cost
- o Tanker Modifications including Ice Strengthening
- o Mooring System Fabrication, Transport, and Installation
- o Riser Fabrication, Transport, and Installation
- o Subsea Templates Fabrication, Transport, Installation and Hookup
- o System Commissioning

In developing costs for the scenarios, the costs associated with crude transport, i.e., shuttle tankers, landbased pipelines, onshore or nearshore terminals, etc., were not included. When comparing fixed and floating systems, it should be recognized that the fixed structure capital requirements include the costs for drilling rigs and support equipment. The floating production system on the other hand, does not include these costs because the rigs are normally leased. In addition costs for such activities such as well completions are much more expensive than those associated with a fixed structure. The difference in cost for Christmas trees is on the order of $650,000 per well. For the 56 well, 100,000 bopd production rate, this amounts to $MM34.5. Drilling times are longer and drilling is more complex through a subsea system. Note also that a significant amount of drilling cost will be incurred prior to commencing production. In the fixed platform case, production can commence on completion of the first well and continue as additional wells are drilled. In summary, when comparing fixed and floating systems it should be borne in mind what is included in each case.

The procedure followed in developing costs and schedules was to initially estimate required material quantities. A schedule was developed based largely on previous industry experience with similar structures. Costs were based on appropriate unit rates for material and fabrication and on prevailing equipment mobilization and day rates for required marine equipment. On completion of cost and schedule summaries a cash flow projection was made for each system. For the specific case of the concrete gravity platform, constructed on the West Coast, a manpower summary was prepared.

To develop realistic cost estimates, a variety of contractors, manufacturers, and equipment suppliers were requested to provide cost information. These firms provided pricing data and current day rates. In addition, BWA and M & R have used their previous experience with similar structures. The information supplied by the various outside sources was compared with our inhouse cost data base and final costs were selected.

The specific assumptions made in costing each system and cost summaries are given in each concept description in Chapters 5 through 12. Total capital costs for the field development scenarios described in Section 3.7 are given in Chapter 12.

5.0 PILED JACKET STRUCTURE

The conventional piled jacket system is the most commonly used platform type for production and production/drilling operations offshore. Over 3,000 of these structures have been placed in the Gulf of Mexico alone. In the design of the jacket structure, the procedure described in Figure 5.0.1 was followed. The present study focused on inplace design of the jacket under wave, ice, and seismic loading conditions, and although provision was made for launching the jackets by inclusion of launch trusses in the weight and cost estimates, no analyses were performed for loadout, transport or launch considerations.

Attention was focused on a platform in 300 ft and 150 ft water depths, which bound the majority of the lease sale area. For base case definition purposes the total deck steel weight and facilities weight was assumed to be 60,000 kips. An additional allowance of 2,500 kips was included for icing, etc., giving a total design base case weight of 62,500 kips.

The design criteria for wave, ice, wind, and seismicity have been defined in Chapter 2. In applying the load conditions, it was assumed that the peak value of each condition acting alone was sufficient to bound the design forces and no joint combinations of ice and wave, ice and earthquake, etc., were applied. In all cases for all water depths both jacket and pile steel was assumed to have a 50 ksi yield stress. The piled jacket was designed according to the provisions of API RP2A, Reference 20.

5.1 Method of Approach

The platform is subjected to wave, ice, and seismic forces while operating inplace. From a preliminary investigation of the relative magnitude of these forces, it was concluded that seismic loads would control the majority of the jacket and pile design. Ice forces were expected to be significant only for the bracing in the waterline region and on the conductor system and wave loads were assumed to have little or no effect. Consistent with these preliminary conclusions it was decided to design the structure for the seismic loading, to check the structure under ice and wave loading and to protect the conductor system from ice using a cage to shield the members.

The design of jacket structures for energy dominated events such as earthquakes, requires some modifications of conventional jacket design practice. In many non seismic regions K bracing is used extensively in jackets. This bracing form is not favored in seismic regions and the X bracing method is more desirable. This bracing type increases structural redundancy and ensures a greater number of potential load paths in the event of brace failure.

Several other measures are also required to increase the ductility of the platform and reduce the possibility of catastropic collapse. The slenderness (kL/R) ratio is maintained at less than 80 for primary members which ensures a predominantly yielding rather than buckling type failure. The main bracing diameter to thickness ratio is held below approximately 38 to minimize difficulties with local buckling. The D/t ratio, however, must also be kept as close as possible to this limit to ensure adequate buoyancy, as tubular members with D/t ratios less than approximately 30 will not float. This, in turn, would increase the need for temporary buoyancy during launch and installation procedures.

The API code, Reference 20 allows the use of more liberal safety factors for design under strength level seismic conditions. A 70% increase (rather than 33%) is allowed for design seismic loading. In the design for pile capacities, the code is less specific. In the present study a safety factor of 1.2 was used on the pile capacities under combined seismic and deck loads. This is consistent with an increase of 70% (rather than 33%) over the normal pile safety factor of 2.0 for operational loads.

5.1.1 Simplified Seismic Analysis

All designs are iterative in nature. The structure is required to establish the loads and the loads are required to design the structure. In the design of the jacket the principal quantities required initially were shear forces and overturning moments. Shear forces control the design of the diagonal members and overturning moments control the number, penetration, wall thickness and diameter of the piles and as a result the required leg diameters. In this study the platform was initially designed for a 300 ft water depth. The intention was to "shrink" the platform by eliminating two bays for the 150 ft water depth. Given the required deck area, slenderness ratios, the need for X bracing and the D/t ratios, and using experience gained in the design of existing platforms in seismic areas, an initial platform layout was derived. The 300 ft platform is shown in Figure 5.1.1 and in the hull size drawing enclosed. A total of eight piled and grouted legs were used to support the deck and facilities, with skirt piles providing increased shear and overturning moment capacity at the base. The platform is continously cross braced, including the region through the waterline, to efficiently transmit the seismic loads.

To initially design the members under seismic loading, a simplified stick model having the same mass and stiffness properties as the real structure was developed, as shown in Figure 5.1.2. The masses were computed by estimating the added and structural masses of the complete platform, Figure 5.1.1. The stiffness was computed by matching second moments of area and the foundation spring stiffnesses were estimated for various pile head conditions using the P/Y curve soil/pile interaction program BMCOL76, Reference 21.

The simplified model was used to establish the shear forces and overturning moments in the jacket. The base case parameters and the range of variation for variables used is given in Table 5.1.1.

The resulting base shear forces and overturning moments for the design response spectrum of Figure 2.3.4 are given in Figures 5.1.3 through 5.1.8. For the base case, in the 300 ft water depth, a base shear of approximately 20,000 kips in the first mode and 8,000 kips in the second mode is indicated. The simplified model can be anticipated to give estimates which decrease in accuracy with increasing mode number. The indicated overturning moment was 7×10^6 kip ft with minimal second mode contribution.

From a review of the simplified model it was clear that the forces increase with the stiffness of either soil translation or rotational springs, increase with increased deck weight and increase with reduced water depth. In the 300 ft water depth the fundamental period was approximately 2 seconds, reducing to approximately 1 second in the stiffer 150 ft water depth configuration.

The simplified stick model provided the required initial data for the member sizes and pile details. It also indicated the potential sensitivity of the concept to changes in deck weight, water depth and soil stiffness. As a result of this study, the pile diameter was set at 60 inches with a leg diameter of 72 inch. A total of 8 main piles and between 8 and 16 skirt piles were projected with a maximum penetration limited to 300 ft. In summary the simplified stick model had provided sufficient information to proceed with a more detailed analysis and design using a realistic structure.

5.2 Platform Analysis

5.2.1 Platform for 300 ft Water Depth

The platform shown in Figure 5.2.1, was analyzed for seismic loads using the response spectrum method of approach and the computer program SACS. A detailed computer model of the jacket was developed, which included all major members. The piles were represented by appropriate short member lengths, computed to have the same stiffness as the spring values derived from P/Y curves and the soil/pile interaction program BMCOL76, Reference 21. In developing the seismic forces, the first 10 modes and periods were combined. The modal combination was through the SRSS method and direction effects were included by applying the 1: 2/3: 1/2 combination rule mandated by API RP2A. A summary of the first ten periods is given in Table 5.2.1. Note the occurrence of several period pairs, indicating relatively close behavior in the two transverse directions.

The pile capacities for the two design soil profiles described in Section 2.5 were computed from the curves shown in Figures 5.2.2 and 5.2.3. Various penetration depths were computed. For a 60 inch pile and a penetration of

300 ft the unfactored capacity is 8,000 kips. Note that a 72 inch pile provides the same capacity with a 250 ft penetration in the weaker soil.

The total resultant base shears and moments developed in the analysis for various directional combinations are shown in Table 5.2.2. For the API mandated combination, a breakdown of base shears and moments by mode are given in Table 5.2.3.

From the response spectrum analysis and the pile capacity curves a total of 24 skirt piles with a 60 inch diameter and 8 main piles with a 60 inch diameter were required. The skirt piles were cut off at elevation - 230 ft as shown on Figure 5.1.1. The piles were arranged as shown on the enclosed full size drawing. It is noted here that for costing purposes, the skirt piles were increased to 84 inch diameter and the number reduced to 16 to reduce the pile installation time required offshore with no significant penalty in terms of environmental load or steel weight. A summary of the pile reactions under both seismic and wave loading for the configuration in Figure 5.1.1 is given in Table 5.2.4.

The member sizes shown in Figure 5.2.1 were derived essentially on the basis of the seismic loads. The platform was checked under the design wave height of 71 ft at periods in the 10 to 18 second range. Wave loads have no significant effect on either the pile design or brace design. A summary of the wave loads on the platform is given in Table 5.2.5. The loads are approximately 10 to 25% of those developed under seismic loading depending on the assumption made in the modelling of the cage system used to protect the conductor system from the ice.

The conductor protection cage is a number of tubular members of 36 inch diameter, and a spacing of 7 ft 6 in., which rings the conductors as shown in Figure 5.2.5 and on the full size drawing enclosed. For design purposes 2 equal groups of conductors were assumed. The closely spaced members present a blocking surface to the wave action and because of the close spacing there will be considerable interference. The lower bound equivalent conductor/cage system was assumed to have the same volume as the sum of the individual conductors. As an upper bound, the system was modelled as two 40 ft diameter cylinders which completely encloses the conductor groups. This assumption leads to a significant increase in shears and moments as shown in Table 5.2.5, but these quantities are still in the order of 20% of the corresponding seismic quantities and no further analysis was deemed necessary for wave loads.

The presence of ice imposes three major additional requirements on the platform.

- o Global Loads
- o Local Waterline Brace Design
- o Conductor Protection

The maximum global loads on the platform are shown in Figure 5.2.6. These loads are in the same order as wave loads and have no effect when compared to seismic loads. The loads were computed based on a 6 ft ice thickness and the pressure area curve of Figure 2.4.4. The ice was allowed to contact the structure from both principal directions. Loads were computed for all members on the face in contact with the structure as shown in Table 5.2.6. In addition, additional loads were added for the effects of the conductor protection cages. It was assumed that only broken ice would contact the cages, and a pressure of 25% of the nominal unbroken ice pressure was used to compute the contribution of these members.

In addition to this computation the possible effects of the platform as whole acting in a blocking mode for the ice was determined. The overall dimensions of the platform in the waterline region are 130 ft x 180 ft approximately. The maximum diagonal dimension is 225 ft. Assuming an effective width equal to the diagonal width of the platform, an ice thickness at the design level for global sheet ice of 2 ft and a pressure of 160 psi, the maximum force on the entire platform is 10,400 kips. This represents less than 30%

of the seismic base shear. In addition the base moment is also approximately 30% of the corresponding seismic value.

The implications are that for all realistic ice conditions, the global effects of ice can be ignored. The local design of the diagonal braces is, however, influenced by contact with the ice. These braces were designed for the concentrated loads imposed by the ice. As a result, braces in the waterline region had wall thicknesses increased by approximately 1/4 inch to resist ice load.

The major influence of the ice loading is on the conductor system. An initial decision was made that unbroken ice would not be allowed to contact the conductor system. A cage, consisting of 36 inch diameter members was used to enclose the conductors. The conductors were divided into two equal groups and the cage with a spacing of 7 ft 6 in. was designed with supports at levels +15, -20, and -90. The cage ensured that no unbroken or significant ice piece can contact a conductor directly.

In summary, an eight legged, 32 pile platform was designed based primarily on seismic loads. Wave loads were found to be insignificant and ice loads could be accounted for by increasing brace thicknesses in the waterline region and by providing a cage for the protection of the conductors.

5.2.2 Platform Design for 150 ft Water Depth

The principles and steps involved in the design of a platform for a 150 ft water depth are the same as those discussed for the 300 ft water depth case. The lower water depth structure could be anticipated to be significantly stiffer than the deepwater case. The simplified stick model discussed in Section 5.1 indicates that the anticipated period in 150 ft water depths is about 50 to 60% of that obtained in 300 ft. The implication of a stiffer structure is increased shear forces and possibly moments from the earthquake.

It was decided to examine two cases for the use of jacket structure in a 150 ft water depth.

1) A single piece jacket carrying all the facilities.
2) Multiple jackets each carrying a share of the facilities.

The single piece jacket was developed by eliminating the upper bracing levels from the 300 ft jacket design. The jacket is shown in Figure 5.2.7 through 5.2.14. The jacket has, as before, 8 main piles at 60 inch diameter and 24 skirt piles an 60 inch diameter. A modal analysis of the platform indicated a fundamental period of 1.24 seconds which is in the expected range. The first 10 periods are given in Table 5.2.7. A full response spectrum analysis was performed on these jackets using the SACS program.

The base shear forces and moments for various directional combinations are given in Table 5.2.8. The shears are approximately 25% higher than the 300 ft water depth case and recognizing the reduced lever arm of the deck mass, the moments are about the same in both cases.

An alternative scheme for field development using multiple jackets was considered. The feasibility of splitting the facilities into separate drilling platform(s), accommodation platform, production platform, utilities platform, pipeline/riser platform, etc., was investigated. The use of such platforms allows a reduction in the deck weight per platform, resulting in a simple system which allows phased development. The multiple platform scheme has an almost guaranteed chance of being technically feasible, as the deck weight can be continuously reduced if required.

The platform layout chosen for the multiple platform scheme is shown in Figure 5.2.15 through 5.2.21. For costing purposes it was assumed that four platforms each carrying 16,000 kips constituted the required platform scheme. The natural periods for the four leg, 12- 60 inch diameter pile platform are given in Table 5.2.9. The base shear and overturning moments as a function of direction are given in Table 5.2.10. The shears and moments are reduced to approximately 35% of those developed in the single platform concept. The piles were designed with a maximum penetration of 300 ft in the weaker soil.

5.2.3 Weight Summary for the Various Cases

The estimated weight breakdown for the platforms in 300 ft, 150 ft single piece and 150 ft multiple structure cases are given in Tables 5.2.11 through 5.2.14. The estimated total weight for the 300 ft water depth single piece structure is 47,900 kips, with the piles/conductors and jacket accounting for roughly equal weights. The estimated weight for the jacket in the 150 ft water depth is 39,500 kips. In the case of the multiple jackets, the estimated weight of a jacket outfitted for drilling is 16,400 kips and without the drilling option the weight is 13,190 kips. The cost summary was prepared for the multiple structure case on the basis of two drilling jackets and two other platforms a comparison of the jacket buoyancy and weight during launch indicates that a minimum of 7.5% excess of buoyancy over air weight is available, with all members closed, indicating that while some temporary buoyancy tanks may be required, the volume will be small and cost insignificant.

5.3 Cost and Schedule Analysis for Piled Jacket Concepts

In this section, the cost and schedule estimates for the piled jacket platform will be presented. Costs are provided for three basic cases:

- Single jacket, all facilities in 300 ft water depth;
- Single jacket, all facilities in 150 ft water depth; and
- Multiple jacket, four jackets carrying all facilities, 150 ft water depth.

Two fabrication sites were considered:

- Japan
- U.S. West Coast

In addition, costs are presented for two production rate cases:

- o 50,000 bopd
- o 100,000 bopd

Only a summary of the cost data is given in this section. An expanded set of calculations is given in Appendix A.

5.3.1 Specific Assumptions

In developing costs for the piled steel jacket platforms the following specific assumptions were included:

- o Material costs were assumed as follows:

  Jacket, Pile, etc., Steel $600/ton, U.S. and Japan
  Module Steel $800/ton, U.S. and Japan

- o Jacket fabrication costs were assumed as $1400/ton in Japan and $1700/ton on U.S. West Coast.

- o Deck fabrication costs were assumed as $1800/ton in Japan and $2100/ton on the U.S. West Coast for the support frame.

- o Module steel fabrication costs were assumed to be $2000/ton in Japan and $2300/ton on the U.S. West Coast.

- o Pile fabrication costs were assumed as $200/ton in Japan and $400/ton on the U.S. West Coast.

- o A weather downtime of 30% was assumed on the time required to set the deck and drive/grout the piles.

- o The derrick barge was assumed mobilized/demobilized to the West Coast for all cases.

- For jacket transport from Japan to Alaska, 2 tugs were assumed. One tug was assumed to be available in Japan and to return to Japan. The transport barge and a second tug were mobilized from the West Coast and returned there. In the West Coast fabrication case, all vessels came from the West Coast, and returned there.

- The deck support frame was assumed towed on one 300 ft x 90 ft barge, with a 6 knot tow to Alaska and all vessels coming from and returning to Japan or the West Coast depending on the fabrication site.

- The piles were carried on two barges with all vessels coming from and returning to the fabrication site general location.

- The deck modules were assumed to be towed in 24 packages on 2 barges, requiring 9 tows, assumed as 4 tandem and 1 single tow. All equipment came from and returned to the general fabrication site region.

- A steam hammer was used for main pile driving and a hydraulic hammer for skirt piles.

- An allowance was made in the development of the costs for the procurement, fabrication, and transportation of the conductors. No allowance was made for the <u>installation</u> of the conductors.

- For the purposes of costing, 8 main piles at 60 inch diameter, and 16 skirt piles at 84 inch diameter were used. The platform was designed with 24# 60 inch diameter skirt piles together with 8 main piles. The 84 inch diameter piles produce equivalent capacity, almost equal material weight, but significantly reduce the pile installation costs and schedule.

- In the development of the cost data, the pile penetrations required for the weaker soil profile have been used both in material fabrication and installation costs.

- The facilities costs defined here for piled jacket structures include the following items.

  - Costs for all topsides production equipment, including oil facilities, gas handling, water/flood equipment, gas compression and lift, storage, generators/electrical, utilities, etc.

  - Cost for drilling equipment including rigs, generators, bulk storage, P tanks, mud tanks, pumps, pipe racks, additional fuel and water storage. Christmas trees are not included.

  - Costs for accommodations as a function of the number of rigs including helideck, and storage of fuel and provisions.

  - Costs for all yard assembly were based on a number of assumptions. The costs were based on a weight of 100% of the dry weight of all production facilities and 70% of the weight of drilling facilities excluding rigs. The fabrication time for the Gulf of Mexico per ton of piping was computed as:

Piping	100 man-hours per ton
Electrical	20 man-hours per ton
Instrumentation	20 man-hours per ton
Miscellaneous	20 man-hours per ton
Total	190 man-hours per ton

Costs were based on the following fully burdened rates:

\$20/manhour, Japan
\$35/manhour, West Coast

In addition a regional cost and productivity factor of 0.9 was used for Japan, 1.1 for the West Coast and 1.0 for the Gulf of Mexico, i.e., a task taking 1.0 hours in the Gulf of Mexico takes 0.9 hours in Japan, etc.

- Costs for offshore hookup were based on a rate of \$50/hour and a conservative estimate of 10% of the manhours required in onshore fabrication.

5.3.2 Costs and Schedules for Various Cases

The costs for a number of different cases for the piled jacket concept are supplied in Tables 5.3.1 through 5.3.6. The 300 ft water depth platform costs have been estimated in Table 5.3.1 for Japanese fabrication and in Table 5.3.2 for U.S. West Coast.

The principal differences in costs are the reduced fabrication costs in Japan being balanced (partly) by the reduced transport/tow cost from the U.S. West Coast. The estimated schedule for construction and installation of the 300 ft platform, fabricated in Japan is given in Figure 5.3.1. Key issues are the pile installation in the limited weather window available. The schedule calls for an approximate 2 1/2 year construction/installation time.

The anticipated costs for a single piece jacket in 150 ft water depth are given in Tables 5.3.3 and 5.3.4 for the two fabrication sites. The costs reflect the reduced weight of material to be fabricated, transported and installed.

For completeness the costs associated with the fabrication of the deck, jacket and piles of the multiple jacket case have been included. These jackets would probably be installed on a staggered basis as the field development evolves. The costs are given for the individual jacket in Tables 5.3.5 and 5.3.6. The cost defined here allows $1,200,000 for the procurement and $400,000 and $800,000 for Japanese and West Coast fabrication of the conductor material, and for its transport to Alaska with the piles. Note that this cost will not be incurred on non-drilling jackets. The total cost of all facilities aboard the assumed four jackets will be approximately the same as for the previous single piece case. A simple field development scenario where all jackets were assumed to be installed individually has been provided in Table 5.3.7 and 5.3.8. In this case the individual jacket costs have been multiplied by four with duplicated conductor costs removed. The usual application of a multiple jacket development is likely to be cheaper than that developed in this simple case, when duplication of mobilization, etc., is removed.

The potential reduction in cost for a lower production rate is discussed briefly in Table 5.3.9. While some modifications in the deck or jacket are likely, these have been ignored. The major impact on cost due to a lower production rate will be due to the reduction in the fabrication and hookup of modules, together with reduced requirements for drilling and production equipment.

5.3.3 Additional Items

A question of significant importance in the application of the piled jacket concept is the time required for installation of the piles. The estimated pile installation time using conventional welding techniques is 139 days including a 30% weather window allowance. The effective time available for pile installation is the 150 day period from May to September, hence, the required installation time is important.

As an alternative, the use of mechanical connectors, in lieu of welding was examined for both main and skirt piles. The results of this study are given in Table 5.3.10. The results indicate that although the use of connectors implies an additional cost of $8.0 MM, the installation time is reduced by 52 days which places the pile installation well within the one season limit. Hence, subject to the acceptability of mechanical connectors, pile installation within a single season can be assured.

5.4 Sensitivity Analysis

A sensitivity analysis was undertaken for four additional parameters together with the water depth. These include:

- Conductor Location
- Deck Weight (Production Rate)
- Soil Profile
- Earthquake Criteria

A fifth sensitivity analysis was conducted for the potential effects of liquefaction and is reported in Chapter 8.

5.4.1 Conductor Location

Two cases were considered, as shown in Figure 5.4.1. In the first case, the conductors were placed in the central region, such that symmetry resulted. In the second analysis, the conductors were placed at one end of the deck. The effect of a nonsymmetric conductor placement is the introduction of potential torsion into the structure. The third platform mode in all cases is torsion. In all cases it was found that placement of the conductors had little effect with minimal changes required in some brace member sizes.

5.4.2 Deck Weight

The effect of deck weight was studied using the simplified stick model of Section 5.1. The ratio of base shears for various ratios of deck weight to the base case deck weight of 62,500 kips (including 2,500 kips for ice accretion, etc.) is shown in Figure 5.4.2 Results indicate that the structural configuration will not change significantly under moderate variations in deck weight and that small changes in deck weight such as would occur for changes in production rate from 50,000 bopd to 100,000 bopd, would have only a minor effect on cost.

5.4.3 Sensitivity to Soil Profile

The soil profile affects mainly the pile design, and to a smaller degree the soil/pile interaction model. Two profiles were considered in this study as described in Section 2.5. Profile A has sand overlying stiff clay and profile B is weaker, with predominantly sand. The basic pile penetration used is 300 ft with Profile B, and for Profile A only 200 ft penetration is required with resulting savings in both steel weight and pile installation costs offshore.

5.4.4 Sensitivity to Seismic Criteria

The seismic criteria specified have a controlling effect on the design of the majority of the jacket members. To assess the effects of using a reduced earthquake level, the API Zone 3 spectrum was compared with the design spectrum used in this study. A comparison between the study design spectrum and various zones specified in the API RP2A is given on Figure 5.4.3. The API Zone 3 spectrum is approximately 45% less than the design spectrum over much of the region of interest.

The three specific cases of the single jacket in 300 ft water depth, and both single and multiple jackets in 150 ft water depths were analyzed. A comparison between the base shears for various earthquake combinations is given in Table 5.4.1. A summary of the comparison of both interior and exterior pile loads is given in Tables 5.4.2 and 5.4.3.

The results indicate that the number of skirt piles can be reduced by 33%. The total estimated weight savings on the jacket system is between 15 and 25%. An estimated weight comparison for the three cases using the design and API spectra is given in Table 5.4.4.

It is clear that the use of reduced seismic criteria would signficantly reduce jacket cost. It should be noted that jacket cost is not a significant percentage of total field development costs. However, the use of reduced design criteria also affects the potential for liquefaction as discussed in Chapter 8.

5.5 Conclusions

The piled steel jacket shown in Figure 5.1.1 is a feasible system. A single piece jacket can be constructed and installed to handle all production and drilling equipment. Specific conclusions and recommendations for the jacket are as follows:

- The jacket member and pile design is controlled by the seismic loading. The loads are considerably reduced by using API Zone 3 rather than the design criteria used in this study. The corresponding weight savings is approximately 25%.

- Ice loading influences the design of the members close to the waterline. The members crossing the waterline require strengthening for local ice forces. In addition, the conductor system must be protected by a cage protection system.

- Wave loads are not critical either in local member or global design.

- The platform schedule is heavily influenced by requirements for pile and module installation and the restricted weather window available. The schedule prepared here was based on the weaker soil profile and the most severe earthquake. Installation times can be improved in the event of:

  - stronger soils
  - lower seismic criteria
  - allowable use of mechanical couplers for the piles.

- The platform behavior is not sensitive to the location of the conductors. Reduction in deck weight has little effect on the jacket but would lead to a decreased number of piles or reduced penetration for the piles.

- Multiple platforms allow a reduction in the complexity of the single jacket. Pile installation on individual jackets is well within the single season limit. In addition the jackets can be custom designed to support designated production and/or drilling equipment. In the present study a single design deck weight of 16,000 kips of facilities and structure was considered.

- The pile jacket structure designed here is conventional in design, fabrication and installation, despite the severe environmental seismic, ice and wave loads. The complexity of the platform falls well within the experience envelope gained with this type of system around the world.

- Material take-off summaries for steel jackets considered in this study are given in Tables 5.2.11 to 5.2.14.

- Costs have been developed for a number of different water depths and production rates. Both West Coast and Japanese fabrication were considered in Tables 5.3.1 through 5.3.10.

- The effects of soil liquefaction on piled jackets must be considered in the design. These are discussed separately in Chapter 8.

6.0 HYBRID STEEL/CONCRETE STRUCTURE

The hybrid structure shown in Figure 3.2.1 combines the well proven jacket concept as the primary support for the deck facilities with a large concrete caisson used for crude oil storage. The design procedure followed was similar to the procedure discussed for the jacket in Section 5. It was assumed that the concrete storage unit would be constructed in a graving dock and floated to an inshore construction site. The steel jacket would be constructed in sections and transported to the inshore site. The sections would be lifted onto the ballasted caisson and grouted and welded into position. The deck and modules would also be installed at the inshore site in a similar manner to the deck installation for the jacket structures. The completed platform would then be towed to site and installed.

In this section we will concentrate on the analysis of the combined steel/concrete system and on the design of the steel upper jacket structure. The concrete base design is described in more detail in the following chapter and will not be repeated here.

In the design of the piled steel jacket, it was clear from the outset that seismic conditions would control. In the hybrid system with its large base caisson, the expected magnitude of seismic and wave load conditions, governing global geotechnical stability at the mudline were such that both conditions required evaluation. At the jacket interface with the base caisson, it was clear that seismic conditions would dominate and that seismic criteria would control the design of the support jacket.

In the design of the jacket, a yield strength of 50 ksi was assumed for the steel. The concrete caisson was assumed to have normal weight concrete with a 28 day compressive strength of 7 ksi and 60 ksi reinforcement steel was used together with standard 270 ksi ultimate strength prestressing steel.

6.1 Method of Approach

The approach used in the analysis and design of the hybrid structure was to separate the design of the jacket and the caisson system. A common caisson was designed for both the hybrid concept of Figure 3.2.1 and the concrete gravity structure of Figure 3.3.1. The details of the concrete gravity structure design are given in Section 7. The base case parameters are as specified for the jacket system in Chapter 5.

6.1.1 Analysis Under Earthquake Conditions

In the analysis of the hybrid system under seismic loading, the approach taken is described in Figure 6.1.1. The design input response spectrum is shown in Figure 2.3.4. This spectrum is calibrated for 5% damping in the various modes. This level of damping is suitable for application in the case of pile structures. It is not applicable for large gravity based caisson systems because of the significantly higher damping inherent in these systems. To ensure a valid comparison between pile structures and gravity structures for equivalent seismic inputs, the following procedure was adopted. Artificial earthquake time histories were generated which yield the same response spectrum as the input design spectrum. A comparison of a typical artificial earthquake shown in Figure 6.1.2 and the target design spectrum (at 5% damping) is shown in Figure 6.1.3. The agreement is excellent.

The resulting time histories were applied to the base of the hybrid structure as shown in Figure 6.1.1. The large concrete caisson modifies the free field seismic behavior of the soil to depths in the order of the characteristic base dimension. This effect called soil/structure interaction is modelled by a series of added soil masses, springs and dashpots. The specific added mass springs and dashpots used in the present analysis were computed by the procedure defined by DnV, Reference 19. The specific spring, and dashpot constants used to represent soil/structure interaction are given in Tables 6.1.1 and 6.1.2. The effective shear modulus used was 1,000 ksf. The large base gravity structure is characterized by significant damping, particularly in translational modes. The critical damping ratio in the present structure is 20% in sway, consistent with DnV Rules where only one half of the theoretically estimated (40%) damping is used.

The derived time history, representing the earthquake was applied to a simplified model for the hybrid structure and its soil structure interaction as shown in Figure 6.1.1. A comparison between the applied input time history in the soil and the time history on top of the caisson, i.e., base of the jacket is shown in Figure 6.1.4. A more illuminating comparison, shown in Figure 6.1.5, is the response spectrum at the top of the caisson (at the appropriate damping ratio for the direction of motion) to the input ground response spectrum (at 5% damping). The comparison give results which are characteristic of most gravity systems. The high frequency, low period motions are severely deamplified. This is due to the inability of the soil/structure system to respond to, and transmit them. The larger period motions are amplified with a peak value at the natural period of the soil/structure system (approximately 2.2 seconds for 300 ft water depths). A careful review of the time histories in Figure 6.1.4 will confirm the shift to longer period motions. The derived motions at the top of the caisson were input directly into the steel jacket as a response spectrum. Given the motions at the caisson top, the jacket can be analyzed using response spectrum techniques in exactly the same way as the piled jacket was analyzed previously except that a fixed base condition is used at the caisson top rather than the pile springs used previously. The SACS program, Reference 37, was used for this analysis also.

6.1.2 Analysis Under Wave Conditions

The analysis of the hybrid system under wave loads is more complex than that previously used for the conventional jacket structure. The wave loading is dominated by the behavior of the base caisson. The platform was modelled using a combination of Morison theory for the jacket portion and diffraction theory on the base caisson. The procedure was simplified by using an equivalent cylinder with height equal to the caisson height and base area equal to the actual square caisson base area. This allows the use of axisymmetric diffraction theory. This simplification gives an excellent representation of the vertical loads on the base caisson and an adequate representation of horizontal load which in no case turned out to be critical to the stability of the platform. BWA's inhouse program EPACS, Reference 38, was used for this computation.

The upper jacket structure was analyzed using the SACS program and the Morison Theory on the undiffracted wave field. The jacket structure contributes only on the order of 5% to the total base shear and jacket member stresses are dominated by seismic rather than wave effects. Hence, the approach taken here is both economical and sufficiently accurate.

6.1.3 Other Considerations

The hybrid structure is completed at an inshore construction site. The complete system must be towed over exposed waters, for a duration depending on the construction location. A preliminary analysis was performed to confirm adequate static stability (GM value) at all stages of both tow and installation conditions. In addition, the deck motions were established under tow for given design seastates. Particular emphasis was placed on establishing whether temporary surface piercing stabilizing columns would be required during tow and installation.

The primary analysis for the hybrid structure was developed for a 300 ft water depth. A 150 ft water depth case was also analyzed in detail. The jacket was developed by eliminating the top two bays of the 300 ft water depth jacket. Additional sensitivity analysis were conducted for varied seismic criteria, production rates and topsides loads.

Ice loading has a similar effect on the hybrid platform as on the piled steel jacket. Global forces were not a concern. The principal effects of ice loading were the design of braces crossing the waterline to resist the ice and the provision of a cage as in Figure 5.1.1 for the jacket to prevent unbroken ice directly accessing the conductors.

6.2 Analysis and Results Summary

The principal analyses conducted on the hybrid structure were for inplace conditions, primarily under seismic and wave conditions.

6.2.1 Seismic and Wave Analysis

The lowest periods of the hybrid jacket structure in 300 ft water depth are given in Table 6.2.1. The motions of the complete platform for 300 ft depth, under the design response spectrum are given in Figures 6.2.1 through 6.2.3. The maximum translational acceleration at the caisson top is 0.12 g, considerably less than the input ground acceleration of 0.35 g. The corresponding vertical acceleration under the design vertical response spectrum (half the horizontal) and the rotational acceleration at the caisson top also given. The parameters of major interest are shown in Figure 6.2.4 through 6.2.6. A maximum base shear of approximately 350,000 kips, with a maximum vertical load of approximately 210,000 kips and an overturning moment of approximately 38 x 10^6 kip ft exists at the mudline.

The maximum wave loads on the base caisson of the hybrid structure for a 300 ft water depth are given in Tables 6.2.2, 6.2.3, and 6.2.4. In the event that wave loads had become critical the wave loads would be recast into an RAO curve and combined with the design seastate spectrum to produce extreme wave force values. An initial review of the wave loads indicated that seismic loads would dominate even if the most extreme load (for period 18 sec) was selected and no spectral analysis was performed. At the aniticipated peak spectral period of 14 seconds, the total horizontal load is approximately 75,000 kips or approximately 20% of the seismic load. The corresponding vertical load is 114,000 kips and the moment is 3.6 x 10^6 kip ft. It should be noted that the moment acting on the caisson alone, is not in phase with the horizontal load. The dominant moment producer on the caisson is the distribution of vertical pressure on the caisson top.

The reduced water depth case considered produced increases in both seismic and wave loads. The lowest periods of the fixed base jacket system are given in Table 6.2.5. A fundamental period of 0.67 seconds was computed. The time histories for base shear, vertical loads and moments are given in Figures 6.2.7 through 6.2.9. The magnitudes are comparable to the 300 ft water depth. The wave loads are significantly higher for the 150 ft water depth case, as shown in a comparison of Tables 6.2.6 through 6.2.8. However, even when the wave loads corresponding to an 18 sec period were considered, seismic loads controlled the base requirements and no detailed spectral analysis was completed for the wave loads at the 150 ft water depth.

The geotechnical analysis of the global stability of this concept was completed in conjunction with the global analysis of the concrete gravity system. The results of this study are described in detail in Chapter 7.

6.2.2 Towing and Installation Analysis

A preliminary study was conducted for the towing and installation of the 300 ft water depth hybrid system. To analyze the towing configuration, a draft of 170 ft was assumed for the tow. It was initially felt that 4 large surface piercing buoyancy columns attached to the corners of the caisson would be required for stability. These columns would be costly and needed to be eliminated if possible. The analysis performed helped reach this objective.

The motions for the combined jacket and base caisson are shown in Figure 6.2.10 for accelerations and Figure 6.2.11 for displacements. The RAOs are given for a location on the corner of the deck. For the design tow seastate specified in Table 6.2.9 the maximum accelerations, velocities and displacements at the most unfavorable corner of the deck are given in Tables 6.2.10 through 6.2.12. The motions are in general moderate with peak accelerations well below those experienced under seismic conditions. The motions are such that no additional buoyancy tanks are required during tow or installation. The hybrid system can be towed at a draft of approximately 170 ft.

The stability of the system during tow and installation was also determined. The metacentric height was estimated over a range of drafts. The critical draft occurs when the caissons top enters the water (120 ft draft for 300 ft water depth case). The minimum GM computed, for this draft, was 11.9 ft indicating that the platform has ample stability even if it had to be deballasted to a shallower draft to clear local obstructions during tow.

6.2.3 Summary of Results

The base case structural details are given in Table 6.4.1. The base caisson has an effective size of 366 ft x 366 ft x 120 ft, with a total volume of concrete of 111,000 cubic yards, and a jacket main framing weight of 14,600 kip. The required storage capacity was assumed at approximately 9 days production or 900,000 barrels for the base case. The jacket layout is given in Figure 6.2.12.

6.3 Cost and Schedule Analysis for the Hybrid Structure

In this section, the cost and schedule estimates for the hybrid platform will be presented. Costs are provided for two basic cases:

- 150 ft water depth
- 300 ft water depth

Two fabrication sites were considered:

- Japan
- U.S. West Coast

In addition, costs are presented for two production site cases:

- 50,000 bopd
- 100,000 bopd

As for the case of piled steel jackets, only a summary of the cost and scheudle data is given in this Section. The basis for the various costs is given in Appendix A.

6.3.1 Specific Assumptions

In developing costs for the hybrid steel/concrete system a number of key assumptions were included. The hybrid system consists of a steel jacket and a concrete storage base caisson. In developing costs, the assumptions made for the steel jacket material and labor costs and for topside facilities are identical to the piled structure case and they are not repeated here.

Specific assumptions used in costing the concrete base caisson were as follows:

- The anticipated average unit costs for various materials are given in Table 6.3.1.
- Both concrete and steel construction were assumed to be either in Japan or in West Coast.
- All components were towed to an inshore mating site with 4 tugs for the base.
- The caisson was ballasted down. The jacket was lifted in 4 sections aboard. Barite ballast was placed to 10 ft.
- The deck structure and modules were lifted and hooked up.
- The entire assembly was towed to location at 170 ft draft.
- The platform was ballasted and undergrouted on site.
- The cost of the site and graving dock required for the base caisson construction was assumed to be spread over 4 users.

6.3.2 Cost and Schedules for Various Cases

The costs for fabrication in Japan are given in Table 6.3.2 for a water depth of 150 ft and in Table 6.3.3 for a depth of 300 ft. The equivalent costs for West Coast fabrication are given in Tables 6.3.4 and 6.3.5. The costs reflect increased fabrication costs on the West Coast and reduced transport/tow charges. Only a cost summary is provided in this report. Detailed cost breakdowns are given in Appendix A under separate cover.

The estimated schedule for the Hybrid structure and the cumulative cash flow for a 300 ft water depth are given in Figure 6.3.1.

The platform would preferably be installed in the June to August period, to minimize effects from seastate and wind conditions. The steel jacket and concrete structure are constructed in parallel. The base caisson is outfitted with all undergrout and ballast equipment, which is tested prior to leaving the graving dock. After completion at the inshore location with the installation of the jacket and modules, and after tow to the installation site, the platform must be set down and the additional heavy ballast introduced to stabilize the structure under design seismic conditions. The base of the caisson will be undergrouted using cement grout. This provides a uniform foundation, avoids hard spot loads and prevents scour under the base.

The effects of reducing the production rate are more significant for both the hybrid and concrete platforms (discussed in Chapter 7) than for the piled steel jacket platform. This is primarily because the reduced production rate implies a requirement for a smaller storage volume in the base caisson, and a smaller mass to be excited under seismic conditions. The anticipated costs for the reduced production rate are given in Tables 6.3.6 and 6.3.7 for the 150 and 300 ft water depth case fabricated in Japan. Both the concrete base and the topside facilities costs are reduced by about 20%.

6.4 Sensitivity Studies

A number of variations of the base case parameters were selected for the purposes of establishing the sensitivity of the concept. In each case only one parameter was varied at a time.

6.4.1 Water Depth Variation

The platform was redesigned for a water depth of 150 ft using the same procedure as before. The top two bays of the 300 ft jacket were eliminated. The base caisson height was reduced to 90 ft in order to keep the caisson out of the wave zone. The resulting platform is shown in Figure 6.4.1. As discussed previously in Section 2, the wave loads for this water depth are significantly higher than for the 300 ft case particularly for longer wave periods. Seismic loads are relatively insensitive to water depth, but in terms of the overall geotechnical stability of the concept, seismic loads dominate.

A comparison of the overall structural dimensions and weight quantities is given in Table 6.4.1. The jacket framing weight required is reduced and the concrete volume required is also reduced. Because of the similarity of the seismic loads, the effective weight of the platform is relatively unchanged with depth and the ballast requirements increase.

6.4.2 Sensitivity to Production Rate

The major impact of changes in the production rate is the resulting reduced demand for storage. The results are summarized in Table 6.4.2. The concrete volume significantly reduces and the effective weight is also reduced. This reduction in mass, however, leads to an increased excitation at the level of the base of the jacket when the earthquake is filtered through the soil/structure springs and the caisson mass. This increased excitation leads to increased forces on the jacket. In conducting this analysis, no allowance was made for the reduction in deck weight implied by reducing the production rate to 50,000 bopd. This implies that the 16,000 kip self weight for the jacket framing is an upper bound.

6.4.3 Sensitivity to the Seismic Criteria

The design requirements for the API Zone 3 and design spectra, Figure 2.3.5 were compared for the hybrid system. The total shear, moment, and vertical loads were significantly reduced. The caisson base size dimensions reduce and the corresponding concrete volumes ballast requirements and effective weight all reduce. The jacket main framing requirements reduce by over 20%. The results are summarized in Table 6.4.3.

6.4.4 Topside Load Variations

The effects of varying deck load in the 40,000 to 100,000 kip range were examined. No significant changes occurred and the jacket weight reduction was estimated at about 5%. Effectively, the overall global behavior of the concept is not sensitive to changes in deck weight.

6.4.5 Variation of the Factor of Safety Used in Sliding

A key question in the design of gravity based systems is the criteria assumed for foundation safety. In resisting environmental loads, a safety factor of 1.5 is usually assumed for sliding considerations. The foundation is designed to have a nominal minimum shear capacity. This criterion is realistic for such steady state loads as ice or loads leading to potentially large movements such as waves. In the case of earthquakes, however, the structural loading is limited by the capability of the soil and the soil/structure system to transmit the load. Unlike wave loading which is a function of structural geometry and is independent of stiffness for fixed structures, seismic loads are a function of system stiffness and increased strength of structures and foundations may be counter productive under seismic loads.

In the design of large gravity based foundation systems under severe earthquakes which occur only infrequently in the life of the platform, a realistic design criterion is the total movement of the structure relative to the soil and the capability of the well and production systems to accept the movement together with the total movement in the soil and the capacity of the foundation after the occurrence of an earthquake. The effective weight required drops by 150,000 tons or approximately 30% in reducing the safety factor on sliding from 1.5 to 1.0. At a safety factor of 1.0 sliding theoretically would just commence. The caisson size required could be reduced even further by allowing sliding to occur, subject of course to minimum volume requirements imposed by oil storage and ballast volumes.

The standard design approach was used in this study. The target factor of safety under seismic loads was maintained at values used for ice and wave type loadings i.e., 1.5 for sliding type failures, 2.0 for bearing/overturning type failures.

6.5 Conclusions

The hybrid steel/concrete structure shown in Figure 6.4.1 is a feasible system. The combined concrete base caisson and steel jacket can be designed to resist the environmental criteria and the system provides integral storage for the required capacity of approximately 9 days production. Specific conclusions and recommendations from this study are:

- The seismic loading dominates the behavior of the platform. Earthquake loads control the design of the jacket and the base dimensions and effective weight required are determined by the seismic loading and criteria imposing standard safety factors on the foundation stability.

- Ice loading affects only the members close to the water line and the conductor system. The members piercing the waterline were strengthened and the conductor system is protected by a cage system as defined in Chapter 5 for the jacket structure.

- Wave loads are not critical either in local member design or in the global geotechnical stability of the foundation.

- The platform can be towed at a draft of 170 ft, without assistance from temporary buoyancy cylinders piercing the surface. For expected seastates, the deck accelerations and motions are moderate and well below the levels experienced during earthquakes.

- The platform can be installed with sufficient positive stability at all times.

- A key issue in the design of the platform is the provision of a 1.5 safety factor on required foundation capacity under earthquake load. A more optimum structure could be developed by relaxing this condition and focusing on the motions of the platform during an earthquake and on the capability of the drilling/production system and the soil system to absorb this movement. An alternative method to design is to limit the platform motions under the earthquake rather than to insist on a factor of safety of 1.5 on foundation capacity.

- Typical dimensions, details and material quantities for the various cases of hybrid structures considered in this study are given in Tables 6.4.1 to 6.4.3.

- Significant savings in cost and material can be achieved by varying design parameters. The 150 ft depth case requires approximately 80% of the total material required for 300 ft. Relaxing spectral criteria to API Zone 3 produces an approximate 30% across the board reduction in all materials. Reduction of the production rate to 50,000 bopd produces a 35% savings in material, primarily in the ballast and concrete because of reduced volume requirements.

- Costs were developed for platforms in 150 and 300 ft water depths, at production rates of 50,000 and 100,000 bopd and for fabrication in Japan and the U.S. West Coast as discussed in Section 6.3.

- Potential liquefaction has a considerable influence on the design of gravity platforms. This is discussed in Section 8.

7.0 CONCRETE GRAVITY STRUCTURE

The concrete gravity structure shown in Figure 3.3.1 is typical of North Sea type applications. The design procedure followed was typical of the procedure used in the design of a jacket. Among the major items considered were:

- Initial establishment of the global dimensions;
- Local design of the base caisson;
- Local design of the concrete towers; and
- Preliminary examination of towing and installation requirements.

From experience with all the fixed structure concepts it was clear that seismic loading would be extremely important. As in the case of the hybrid steel/concrete concept, it was not known initially, however, how significant the wave loads would be, particularly in terms of the overall geotechnical stability. From initial analysis it was clear that seismic considerations would dominate the tower design and their interface with the base caisson.

In the design of the concrete structure the material properties listed in Table 7.0.1 were used. Normal weight concrete was used with a 28 day strength of 7 ksi. Standard values of 60 ksi yield and 270 ksi ultimate strengths were used for the reinforcing and prestressing, respectively.

7.1 Method of Approach

The method of approach used to design the concrete tower was to initially establish the global layout of the platform. As a base case the conditions defined in Table 7.1.1 were used for the initial design. The structural layout chosen is given in Figure 7.1.1. The freeboard was chosen at 75 ft. This accounted for the extreme wave crest height, tides and surge, reservoir subsidence, settlement, caisson effect and air gap as set forth in the DnV regulations, Reference 19. To accomodate the deck load, and the facilities, a four tower structure was chosen. Two of the towers were sized to accommodate 29 conductors each and the remaining two towers accommodating various utilities were given sizes typical of North Sea gravity platforms.

The caisson size was developed to accommodate at least 9 days production (900,000 or 450,000 barrels). Other considerations in establishing the caisson size were volume for ballast during towing and operational phases, adequate foundation resistance and a length to breadth ratio and a height to water depth ratio typical in North Sea Applications. The caisson height of 120 ft is 40% of the water depth which is typical of North Sea platforms.

7.1.1 Methods of Analysis

The procedures used in the design of the concrete gravity structure parallel those used in the design of the hybrid steel/concrete system described in Section 6.1. Seismic global analysis was performed as described previously. Since the base caisson is identical in both systems, the soil/structure interaction springs, dashpots and masses are identical. As described in Section 6.1.1 the seismic loads were filtered through the base and the soil/structure system to the tower/base intersection. The input excitation at the top center of the base caisson was applied to the towers as shown in Figure 7.1.2. The towers were analyzed as a series of beam elements. The total moments and shear forces were computed using the SRSS method and the API directional combination 1:2/3:1/2. The applied axial loads and moments were used to design the towers.

The base caisson was designed using simplified standard methods of analysis. The outer walls were designed to resist implosion according to the DnV Rules, Reference 19. Interior Walls, Base and Top Slabs were designed for the appropriate hydrostatic head. The wave load analysis was performed as a combination of a diffraction analysis on the base caisson and a Morison analysis on the undiffracted wave field for the towers, as described for the Hybrid system in Chapter 6.

7.2 Analysis and Results Summary

The principal loading condition on the concrete gravity system is produced by the design response spectrum. Wave loads are significant, but seismic conditions still dominate. A comparison of the maximum shear and moments at the tower base is given in Table 7.2.1. The towers were proportioned as shown in Figure 7.2.1. The initial selection for diameter to thickness ratios was based on North Sea practice. The towers were tapered to reduce weight, consistent with the required strength.

The towers were analyzed and designed under the design moment using the BWA nonlinear design program RECODE 2, Reference 18. The program has full provision for nonlinear behavior of the concrete, steel, and prestressing. Cracking is fully included. A typical design condition is given in Table 7.2.2 together with the concrete, steel, and prestressing requirements. The towers were also designed for a maximum shear force of 7 * SQRT (f'_c) , consistent with the recommendations for cyclically loaded shear walls described in Reference 25.

The base caisson was designed for a series of hydrostatic conditions listed in Table 7.2.3. Inplace hydrostatic pressures were assumed to be those occurring at installation or at the setting of the deck as appropriate. Interior walls were designed for potential damage conditions during deck setting, towing, and installation conditions. Appropriate safety factors were applied as required by the ACI procedures.

The static stability of the platform during deck setting and towing were established. Deck setting was assumed to occur with 20 ft of tower freeboard. A total of 10 ft of barite ballast coupled with 26.3 ft of water was required for towing conditions. The ocean tow displacement was 522,700 tons at a draft of 150 ft. This draft can be increased as required. The minimum GM was computed as 2.0 ft at the critical draft of 120 ft, when the caisson top is just submerged. The platform effective weight is 861,000 kips in operation, with a total of 32 ft barite ballast required.

A summary of the key base case dimensions is given in Table 7.2.4. The platform has the conductor system protected by the legs and no special ice protection is required as is the case with the other two concepts, although the concrete legs shear capacity is increased locally to withstand contact with the drifing ice.

7.3 Global Geotechnical Analysis of the Hybrid and Concrete Gravity Systems

The global geotechnical analyses for the hybrid structure described in Chapter 6 and the concrete gravity platform are identical. The structures have essentially the same global seismic and wave loads, since the base caisson and deck weight contribute most of these loads. The approach taken, was to develop a set of loading conditions which bounded the expected conditions for both structures. These loading conditions are given in Table 7.3.1, for both the 150 ft water depth and 300 ft water depth cases. A very conservative period of 18 seconds was used with the design wave height. Since a deterministic wave at this single long period, did not control the design, no further more detailed analysis using the design wave spectrum was performed.

The platforms were checked initially for bearing capacity as shown in Figure 7.3.1 and 7.3.2 for both design profiles given in Figure 2.5.1. A factor of safety in excess of 2 was found in all cases. Two sliding modes were also checked including the shallow sliding mode of Figure 7.3.3. The factors of safety for the various loads are given in Table 7.3.2. All safety factors exceed 1.5 as required.

Finally a deep sliding mode was considered for each soil as shown in Figure 7.3.4 and 7.3.5. Factors of safety of approximately 1.3 are developed in this analysis as given in Tables 7.3.3 and 7.3.4 which are less than 1.5. The implications of requiring a safety factor of 1.5 has been discussed extensively in Section 6.4. To achieve a factor of safety of 1.5 in the deep sliding mode, the base dimensions must be increased by approximtely 50 ft in each direction. To maintain equal volume, the height can be reduced to approximately 90 ft. The impact on cost or schedule is minimal.

7.4 Cost and Schedule Analysis for the Concrete Gravity Platform

The cost and schedule information for the concrete gravity platform are provided in this Section. Specific costs will be provided for a number of basic cases including:

- o water depths of 150 ft and 300 ft
- o fabrication on the U.S. West Coast and Japan
- o production rates of 50,000 and 100,000 bopd

The concrete structure cost and schedule assumptions are identical to those for the Hybrid structure given in Chapter 6. Refer also to Appendix A.

7.4.1 Costs and Schedules

The costs for a number of cases are given in Table 7.4.1 through 7.4.4. Costs for Japanese fabrication in two water depths at a production rate of 100,000 bopd are given in Tables 7.4.1 and 7.4.2. The corresponding schedule and cash flow for the 100,000 bopd production rates are given in Figure 7.4.1. Costs for West Coast fabrication are given in Tables 7.4.3 and 7.4.4. The appropriate costs reduction for a 50,000 bopd production rate are given in Tables 7.4.5 and 7.4.6 for water depths of 150 and 300 ft and fabrication in Japan. Finally a manpower summary for West Coast fabrication is given in Figure 7.4.2.

The dominating influences and methodology in the fabrication and installation of the concrete gravity platform are as described in Chapter 6.

The fabrication and installation schedule for the concrete gravity platform, fabricated in Japan for 300 ft water depths and 100,000 bopd is given in Figure 7.4.1. The required cumulative cash flow is also provided.

7.5 Sensitivity Analysis

Two principal sensitivity cases were considered as shown in Table 7.5.1. The water depth was reduced to 150 ft and the platform configuration is as shown in Figures 7.5.1 and 7.5.2. The base caisson was reduced to 80 ft height for the same reasons as discussed in the Hybrid Concept. The analysis procedures were identical to those described previously for the 300 ft case. A summary of the significant dimensions is compared with the case in Table

7.5.2. The principal changes are an increased base area, but a reduced height, concrete volume and a similar effective weight. The maximum base shear is essentially the same as the 300 ft case, which indicates a requirement for a similar effective weight and consequently an increased ballast.

The second condition considered was a reduction to 50,000 bopd production rate. The reduced storage volume allows a reduced caisson size. The base shear reduces by approximately 20% which allows a similar reduction in effective weight and consequently a comparable savings in barite ballast.

7.6 Conclusions

The concrete gravity platform is a feasible system for application in the North Aleutian Basin. The platform has the capability of resisting all the applied environmental loads during its lifetime. Experience with this type of platform in the North Sea has shown it to be a realistic candidate for consideration as a production system in the North Aleutian Basin. Specific conclusions of this study are:

- The dominant environmental load is produced by the design seismic response spectrum. As in the case of the hybrid system the base soil/structure system significantly reduces the high frequency content of the earthquake and the 20% damping controls amplifications around the natural period. Seismic loads control both the design of the towers and the global dimensions of the base caisson.

- The base caisson was designed for all applicable hydrostatic conditions. The wall thicknesses and layout derived are typical of North Sea gravity structures.

- Wave conditions are more severe on the gravity platform than is the case with the other concepts. However, they influence only such aspects as deck elevation and do not control member or global dimensions.

- Ice loading is not severe either in the local or global sense. Because drilling is performed through the legs no specific ice cage protection is required for the conductors other than increased shear stirrups in the waterline area.

- Factors of safety of at least 1.5 were achieved for most sliding conditions and factors exceeded 2.0 for all bearing conditions checked. For some deep sliding modes the safety factor is approximately 1.3. The safety factor can be raised to 1.5 with some minor adjustments to the global dimensions of the base caisson with no significant effect on cost.

- Typical dimensions, details and material quantities for the various cases of concrete gravity platforms considered in this study are given in Table 7.5.2.

- The platform has sufficient floating stability for all required mating, towing, and installation.

- Cost, schedule and manpower information for various production rates and water depths is provided in Section 7.4 and the associated Tables and Figures.

- Liquefaction has a significant effect on the behavior of concrete gravity platforms. The effects are discussed separately in Chapter 8.

8.0 LIQUEFACTION POTENTIAL ANALYSIS

Liquefaction refers to a phenomenon in which a cohesionless soil undergoes an increase in internal pore water pressure leading to a decrease in effective stresses and corresponding loss of strength when it is subjected to cyclic loading, such as during an earthquake.

The North Aleutian area is located in a very seismically active region. Since the soil conditions include large deposits of cohesionless soil, the potential for liquefaction under seismic loads has been evaluated. This section presents an estimate of the liquefaction potential for a range of typical soils encountered in the area of interest. It includes a brief description of the methodology, as well as the results and conclusions.

8.1 Method of Approach

Soil liquefaction potential under earthquake loading has been studied under two different conditions: 1) Free-field conditions, i.e., the soil in its natural state without any structure placed on it; 2) Under the conditions imposed by a structure placed on the soil deposit. The stress conditions in the soil due to a piled platform have been assumed to be similar to those under free-field conditions, since most of the soil will not experience any significant change in its stress state (except near the piles). On the other hand, the presence of a gravity structure will significantly change the initial stress state. Figure 8.1.1 shows the soil under free-field conditions, under a piled structure and under a gravity structure.

Liquefaction potential analyses were conducted using two approaches:

- Assuming undrained conditions;
- Accounting for redistribution and dissipation of pore pressures.

The liquefaction potential was evaluated for a deposit of granular soil, with thickness of 60 ft and effective unit weight of 60 pcf; this corresponds to the soil conditions of Profile A. The cyclic resistance of the soil was obtained from a report by Ertec (1983), Reference 31, which included soil data and results from cyclic direct simple shear tests on four different types of soil recovered in the North Aleutian area. The results from these tests are presented in Figure 8.1.2; the soil cyclic resistance is expressed as the ratio between the cyclic shearing stress and the effective initial vertical stress. In this study, the complete range of cyclic resistances has been covered by considering the lower and upper bound curves, as shown in the figure.

Figure 8.1.3 shows the range of grain size distributions for the four types of soil that were tested.

8.2 Analysis and Results

8.2.1 Undrained Conditions

a) Free-field Conditions:

Analysis of liquefaction potential under undrained free-field conditions was accomplished using the simplified method proposed by Seed and Idriss (1971) Reference 32.

The actual time history of shear stress in a soil deposit during an earthquake will have an irregular form; in this method, such irregular history is converted into an equivalent cyclic shear history, with an equivalent number of cycles (N_{eq}) of uniform average shear stress (S_{av}).

The basic steps in this approach are:

1) Computation of maximum shear stress due to the earthquake at different depths within the soil:

$$S_{max} = (wh/g)\, a_{max}\, r_d \qquad (8.2.1)$$

where:

S_{max}	=	maximum shear stress due to the earthquake;
w	=	total unit weight of the soil;
h	=	depth below ground surface;
g	=	acceleration of gravity;
a_{max}	=	maximum ground surface acceleration;
r_d	=	stress reduction coefficient with a value less than 1.0.

The stress reduction coefficient r_d accounts for the deformability of the soil (if the soil behaved as a rigid body, r_d would be equal to 1.0). Seed and Idriss presented a range of values of r_d for different soil profiles, which is shown in Figure 8.2.1. Since the scatter of the results in the upper 30 to 40 ft is not great, Seed and Idriss recommend the use of the average r_d, shown by the dashed line in the figure.

2) Determination of the equivalent uniform average cyclic shear stress at different depths: based on laboratory data, and through a method of weighting individual stress cycles, Seed and Idriss propose that the average equivalent uniform shear stress S_{av} is about 65 percent of the maximum shear stress S_{max}.

3) Determination of equivalent number of significant stress cycles: the appropriate number of significant stress cycles N_{eq} depends on the duration of ground shaking, i.e., on the earthquake magnitude. Seed et al (1975, Reference 33), present a relationship between earthquake magnitude and a range of equivalent number of cycles, which is shown in Figure 8.2.2. The mean values from this figure have been used in the study.

4) Determination of stresses causing liquefaction after the same number of stress cycles: this was accomplished using the results

of the cyclic direct simple shear tests, which had been presented in Figure 8.1.2. As indicated in Section 8.1, this was done for both the lower and upper limits of the cyclic resistance range shown in Figure 8.1.2.

5) Evaluation of liquefaction potential: this was accomplished by comparing the profiles of stress causing liquefaction (from Step 4) and equivalent uniform stress induced by the earthquake (from Step 2).

The results for free-field conditions in the soil are presented in Figures 8.2.3 and 8.2.4, corresponding to the lower and upper bounds, respectively. In these figures, the profiles of cyclic shear resistance for three earthquake magnitudes are compared to the induced cyclic shear stress profile, corresponding to a peak ground acceleration of 0.35 g; this peak acceleration was derived from an artificial earthquake, as discussed in Sections 2 and 6. The three earthquake magnitudes (6, 6 1/2 and 7) cover a range of probable magnitudes for the North Aleutian Area. In these figures the cyclic shear stresses have been normalized with respect to the initial effective vertical stress.

In Figure 8.2.3, which corresponds to the lower-bound cyclic resistance, the shear resistance for an earthquake of magnitude 7 is less than the earthquake-induced shear at any depth, therefore, liquefaction is likely to occur in the whole deposit. On the other hand, for an earthquake of magnitude 6 (with the same peak ground acceleration of 0.35 g), liquefaction is likely to take place only in the upper 40 ft of the deposit.

If the upper-bound cyclic resistance is used (Figure 8.2.4), an earthquake of magnitude 7 might cause liquefaction in the upper 35 ft of soil, while a lower magnitude earthquake probably will not result in liquefaction.

b) <u>Under Gravity Structure</u>

Estimates of liquefaction potential under gravity structures have been evaluated using the same simplified method by Seed and Idriss (1971), Reference 32, as described in Section 8.2.1a.

The major difference between this case and the previous case arises from the changes in the induced stresses on the soil due to the gravity structure. First, the structure will apply a confining pressure to the soil underneath it. Second, the induced shear stress in the soils will depend on the dynamics of the structure.

The peak shear force at the soil-structure interface can be calculated as follows:

$$T = m_{total} * a_{max} \qquad (8.2.2)$$

where:

T = peak shear force;

m_{total} = total mass, including structure and added mass (added mass = mass of displaced water multiplied by an added-mass coefficient);

a_{max} = peak structure acceleration.

The resulting peak shear stress at the soil-structure interface is:

$$S_{max} = T/A \qquad (8.2.3)$$

where:

A = base area of gravity structure.

At any depth within the soil deposit, the equivalent uniform average cyclic stress can be estimated adding the stress due to the gravity structure, and the stress due to the soil column above that depth; the average cyclic stress is assumed to be 65 percent of the peak, i.e.:

$$S_{av} = 0.65\ [(m_{total} * a_{max}/A) + (wh/g) * a_{max}]r_d \qquad (8.2.4)$$

An explanation of the use of the stress reduction coefficient r_d was presented in Section 8.2.1.a. Figure 8.2.1 illustrates the range of values of r_d vs. depth, as well as the recommended average values of r_d to use with Equation 8.2.5.

The results for conditions under a gravity structure, such as the hybrid structure in Chapter 6 and the concrete platform in Chapter 7, are presented in Figures 8.2.5 and 8.2.6. These platforms impose an effective confining stress of 4.5 ksf. The peak acceleration in the presence of the gravity structure was estimated through a soil-structure interaction analysis, as presented in Sections 6 and 7. The shear stresses were calculated assuming an added-mass coefficient of 1.0.

Figure 8.2.5 shows the results using the lower-bound cyclic resistance; this figure indicates that for an earthquake of magnitude 7, liquefaction is likely to occur in the upper 30 ft of soil, while for an earthquake of magnitude 6, liquefaction might not occur anywhere in the deposit. On the other hand, using the upper-bound cyclic resistances, liquefaction is unlikely to take place, even for the stronger earthquakes of magnitude 7; this is illustrated in Figure 8.2.6. From Figures 8.2.3 through 8.2.6, it can be concluded that the presence of a gravity structure greatly reduces the liquefaction potential of the soil under the structure.

Note that Figures 8.2.5 and 8.2.6 illustrate the results for the soil directly under the gravity structure. Conditions in the soil outside this area are different; at a reasonable distance away from the structure, the free field condition applies.

8.2.2 Redistribution and Dissipation of Excess Pore Pressures

The effects of redistribution and dissipation of pore pressures on the liquefaction potential of a soil mass may be quite significant. For example, if the pore-water pressures generated are to some extent dissipated, then liquefaction may be averted; conversely, the dissipation of pore-water pressures generated deep within a soil mass may result in upward seepage and consequent liquefaction of surface layers (Booker, Rahman, and Seed, 1976, Reference 34).

Seed, Martin and Lysmer (1976, Reference 35) presented a method to analyze the development and redistribution of pore-water pressures in a horizontally stratified deposit of sand. Using the same basic equations governing the generation and dissipation of pore pressures, Booker et al, developed a method of analysis based on the finite element method; the problem is solved with the aid of the computer program GADFLEA (Booker, Rahman and Seed, 1976, Reference 34). This computer program can account for variations in the coefficient of volume compressibility due to changes in the excess pore pressure; input data include the geometry of the problem (including drainage conditions), finite element mesh, stresses at desired points within the soil deposit, values of coefficient of permeability, earthquake data and cyclic soil resistance; the computer program calculates excess pore pressures at each element node, for any specified time, during and after the earthquake.

Figures 8.2.7 and 8.2.8 show the results of the case using the lower-bound resistance, and assuming the coefficients of permeability in the vertical and horizontal directions (k_v, k_h) to be equal to 0.328×10^{-3} ft/sec; the coefficient of volume compressibility m_v was assumed to be 1.0×10^{-6} ft^2/lb; these values of k_v, k_h and m_v are typical for this type of soil, as indicated by Seed et al (1976, Reference 35). The figures illustrate the excess pore pressure ratios due to an earthquake of magnitude 7, whose duration is 25 seconds. The results correspond to the soil directly under the gravity structure. The onset of liquefaction is denoted by an excess pore pressure ratio of 1.0 (when the excess pore pressure is equal to the initial effective vertical stress). Therefore, liquefaction is predicted to occur in the upper 30 ft of the deposit, beginning about 20 seconds after the earthquake starts.

In order to investigate the effect of the coefficient of permeability, two additional cases were studied, the results of which are shown in Figure 8.2.9. An increase in permeability by one order of magnitude (to 0.328×10^{-2} ft/sec) would result in considerable lower excess pore pressure ratios, and an increase by two orders of magnitude (to 0.328×10^{-1} ft/sec) will result in almost negligible values of excess pore pressure ratio. Therefore, if the permeability of the soil is greater or if it could be increased (say, through the use of gravel drains or by other means), the potential for liquefaction would be significantly reduced.

The cyclic resistance of the soil depends on its previous history; if for example, the soil under a gravity structure has been subjected to cyclic loading which do not cause failure, and the resulting excess pore pressures dissipate, the soil will experience an increase in cyclic strength. This can be illustrated in Figure 8.2.10. This figure shows the initial cyclic resistance of a sand from the Ekofisk tank area, with a relative density of 77%, as well as the cyclic resistance of the same soil after four small storms had been simulated, and the resulting excess pore pressures dissipated (Lee and Focht, 1975, Reference 36). The results clearly indicate that the four storms had a significant effect in the cyclic strength of this sand.

In Figure 8.2.10, the curves corresponding to the Ekofisk sand are presented together with the curves for the North Aleutian area. When the number of stress cycles is small, as for the earthquake magnitude considered in our analysis), the initial strength of the Ekofisk sand is very close to the lower bound for the North Aleutian. Therefore, in the following analysis, the behavior of the Ekofisk sand will be assumed to be representative of the sands in the North Aleutian area.

Using the cyclic resistances from the Ekofisk tank, the behavior of the sand under the gravity structure is presented in Figure 8.2.11. This figure corresponds to the same earthquake, i.e., magnitude 7 and 25 seconds duration. The results show that the previous four storms clearly reduce the liquefaction potential. This was not taken into account in the Ertec test results.

8.3 Implications for the Various Concepts

The implications of soil liquefaction vary for the various concepts described in this study. The floating systems are the least affected by the occurrence of liquefaction. Piled jacket structures are affected by a loss of support over a limited length of pile causing a reduction in the axial capacity, but more importantly, significantly increasing the bending moments for a given constant load. The most severely affected concepts are the gravity based hybrid and concrete systems described in Chapters 6 and 7.

In this section the potential consequences of liquefaction on the various concepts will be discussed.

8.3.1 Piled Jacket System

To simulate the effects of liquefaction on the pile system, the platform model was revised as shown in Figure 8.3.1. It was assumed that the top 30 ft of soil was removed. Two cases were studied under seismic loading:

- The single jacket in 300 ft of water;
- The multiple jacket case in 150 ft of water.

The response spectrum analysis indicated that while the unsupported pile length increased the bending in the individual piles and in the lower jacket bay braces, the increased flexibility of the jacket reduces the total base shears and moments.

The lowest natural periods for the various cases are defined in Tables 8.3.1 and 8.3.2. There is a significant increase in the flexibility of the jacket under the design earthquake. A comparison of the RMS base shears for cases with and without liquefaction is given in Table 8.3.3. A summary of axial loads in both interior and exterior piles for the 300 ft water depth is given in Table 8.3.4.

The major impact of liquefaction is on the lower bay diagonal braces and on the piles. The lower braces require an increased thickness in both water depths and the skirt pile diameters must be increased to 84 inches from 60 inches. In addition, the leg thickness needs to be increased in the lower bay for the 300 ft water depth case.

An estimate of the required steel weight increases for the various cases is given in Table 8.3.5 through 8.3.7. The results indicate an increase of 10% in the required steel weights. The multiple jacket weight given in Table 8.3.7 are for the drilling jacket case. A similar increase of 10% is required for the production, etc., jackets in this multiple jacket case.

8.3.2 Gravity Platforms

If the soil under a gravity structure should liquefy, the effects on the structure could be severe. The increase in excess pore pressures would result in a reduction of effective stress, and could lead to foundation failure. Depending on the soil conditions and magnitude of this earthquake, the structure could experience a significant amount of settlement, accompanied by severe tilting; also, lateral movements of the structure could occur. However, as indicated in Section 8.2, the presence of the structure significantly reduces the liquefaction potential of the soil underneath the structure. Liquefaction potential can also be reduced if the soil permeability is increased, say by the use of gravel drains.

8.3.3 Floating Systems

The floating systems are the least affected if soil liquefaction occurs. The major effect would be a reduction of the mooring system capacity during an earthquake and possibly an increased mooring capacity after the event if the anchor were to sink into the liquefied seabed soil.

8.4 Conclusions and Recommendations

Based on the liquefaction potential analysis study, the following conclusions can be drawn:

1) In general, soil liquefaction potential depends on the soil properties, stress conditions in the soil mass, and magnitude and duration of the cyclic loads induced by the earthquake.

2) The soil conditions in the North Aleutian area are variable from site to site. In some cases, liquefaction will not be a problem. However, liquefaction may pose a severe problem at a number of sites.

3) Piled structures may not always rely on support from the upper layer of cohesionless soil. The piles should, therefore, be designed accordingly.

4) If the lower-bound cyclic resistance is used with no consideration to loading history, the study shows that the soil under the gravity structure might liquefy under a strong earthquake. This is true even if the dissipation of pore pressures during the earthquake is accounted for. However, if there is an increase in soil permeability (for example, with the aid of gravel drains in the soil), liquefaction can be averted.

5) The potential for liquefaction of the soil under a gravity structure is reduced by previous cyclic loading (such as that due to storms or smaller earthquakes). When this is taken into account, the liquefaction potential is significantly reduced. The Ertec test results did not consider this factor.

The results of the study indicate that for certain soil conditions, a strong earthquake will not cause liquefaction of the soil underneath a gravity structure. However, for more unfavorable soil properties, liquefaction could take place, unless the soil permeability is increased (for example, through the use of gravel drains).

It is important to point out that the analyses were based on cyclic resistance tests conducted on soil samples obtained from a wide area of the North Aleutian Shelf. These samples may not accurately represent the field conditons due to factors such as sampling disturbance. Once the structure location is better defined, a sampling and testing program can be conducted to determine the actual field conditions and to estimate the appropriate cyclic resistance of the soil. The test program should concentrate on reducing the effects of sample disturbance; and prior loading. Better data on these two factors can significantly reduce the predicted liquefaction potential.

9.0 FLOATING PRODUCTION, STORAGE, AND OFFLOADING SYSTEMS

The selected concept for the FPSO and FSO systems consists of a turret moored tanker with a flexible riser system. The rationale for this selection was discussed in Chapter 3. A symmetric, eight leg, all chain mooring system enables the tanker to maintain position over the subsea systems. The details of this concept are applicable to both the FPSO and the FSO systems. In all floating systems a single target water depth of 250 ft was considered.

9.1 Method of Approach

Once the final concept was selected, a final pass was made to determine the particulars of the major systems in the concept. The major systems finalized were: mooring system, riser system, and the swivel. Additional investigations were made into the deicing and subsea systems. A fully detailed design of these systems is not possible unless the exact vessel and field particulars are known, but the design developed here is consistent with the objectives of this study.

The mooring system design was finalized using a BWA static mooring program, DAMS, Reference 22, which gives line force data as a function of excursion. The mean static force the system must resist was computed as 760 kips. This force is generated by the one hundred year storm acting on the 126,000 DWT captive tanker. No ice loads were considered to act in conjunction with this storm loading. The mooring system must also withstand a first order surge motion of 14 ft and a computed slowly varying drift force of 100 kips.

The mooring system has been designed to sustain ice loads, acting alone, generated under expected conditions in the region. The anticipated ice features for mooring system design are small floes ranging in thickness from 2 ft to 4 ft. Additionally, it was assumed that the tanker is always bow on to the ice. This assumption is reasonable since thrusters can be used to maintain this orientation. It should be noted also that ice conditions are intermittent in the region and that in many years no ice occurs over much of the area.

The flexible riser system must be designed with due consideration given to the mooring system. Since the mooring system has eight legs, symmetrically spaced, the riser system can have no more than eight risers. This numerical restriction is imposed to eliminate interference problems between the risers and the mooring lines.

Of the eight flexible risers, seven are used for well production and control. The eighth riser is used for gas reinjection. These are multibore risers, where the product test annulus and control lines are integrated into a single bundle. This type of riser is still in the test/development stage and hence, has no track record. However, the technology necessary for the development of this system does not present major problems and this system will be applied in the future.

For the FSO system, only a single flexible product line is necessary. This type of riser has been used on several installations worldwide and is a proven concept.

The FPSO and FSO tanker systems were designed with limited ice strengthening. In the cost analysis for these systems, an allowance was made for the cost of ice strengthening to allow the tankers to maintain station under all expected ice conditions without significant hull damage.

The flexible riser system for the FPSO enters the tanker turret and terminates in a manifold assembly. The purpose of the manifold is to reduce the number of passes required in the swivel. The high pressure swivel enables the flow lines to exit the turret onto the weather vaning tanker. For the FSO system, the manifolding is not necessary and a much simpler swivel can be used.

The subsea end of the riser connects to a pipeline end manifold, or PLEM. The PLEM serves as an anchor for both the riser and the subsea pipelines. The subsea pipelines extend from the PLEM for approximately two miles, toward the subsea templates. Just before the template, the pipeline terminates at the flowline anchor base, or FLAB. The FLAB serves to anchor the other end of the pipeline, and to relieve the template of thermal and earthquake loads induced by the pipeline. A short spool piece connects the FLAB to the subsea template.

The last major system studied was the deicing system. Since the lease sale area is in a severe icing zone, it was anticipated that a large amount of ice could accumulate on the tanker if counter measures were not taken. In order to prevent icing of the process equipment, a protective cover will be built around it. This will provide a smooth, flat surface to reduce ice accumulation and make the structure easier to deice. There are two drawbacks to this solution; firstly, the center of gravity of the vessel is raised, and secondly, the wind loading on the vessel is slightly increased. Neither effect poses significant problems.

Of the various deicing methods available today, the thermal methods have had the best success. One thermal method, the thermosyphon, has been applied on a large scale to a Japanese ship. This method involves routing heating pipes under the exposed surface areas. Another possible thermal solution is to electrically deice the structure. The boilers on the tanker will have enough capacity to supply the necessary power for the process equipment and the deicing mechanism, even at the extreme icing rate.

9.2 Analysis Results and Evaluation

Using 5 3/4 inch diameter, grade 3 chain the required mean static force of 760 kips for the FPSO/FSO is developed at an offset of 26 ft. The first order surge of 14 ft brings the total offset to 40 ft. At this offset, the tension in the most loaded line is 1,350 kips and the system restoring force is 1,975 kips. Adding a slowly varying drift force of 100 kips brings the total system restoring force to 2,075 kips, the total excursion to 40.5 ft, and the tension in the most loaded line to 1,450 kips. This tension load is 48% of the catalog breaking strength of the chain, and hence, meets the guidlines (50%) set forth in API-RP2P, Reference 23. Figure 3.4.8 depicts this system.

It should be noted, that the parameter that has the most significant effect on the mooring system design is the first order surge motion of the tanker. Other important parameters affecting the mooring system are: water depth tanker draft and wave height. The loads developed under expected ice conditions are less than those experienced in the 100 year storm. The loads developed under expected ice conditions are less than those experienced in the 100 year storm.

For the seven production risers used in this study each riser produces approximtely 14,000 barrels of oil per day. This flow requires an internal diameter of 4.6 inches based on API-RP14E, Reference 24. The test and annulus lines only have to support one well at a time so their diameter of 3 inches is adequate.

The final flexible production riser, shown in Figure 9.2.1, consists of 1-5 inch product line, 2-3 inch lines for annulus and test, and 3-1 inch lines for control and injection. The entire bundle has a diameter of approximately 15 inches and is 62% buoyant. The flexible gas reinjection riser requires an internal diameter of 8 inches and a corresponding outer diameter of 11.5 inches.

The upper end of the risers terminates in the manifold system beneath the swivel. The lower end of the risers terminate in the PLEM. The location of the PLEM affects the length of both the risers and the subsea pipelines. More importantly, however, the location of the PLEM has a major affect on the dynamic interferance between the risers and the mooring chains. In order to reduce this problem, the PLEM's have been placed inside the circle formed by the mooring chain touch down points.

For the FSO system, only a single flexible riser is required to conduct the production from the fixed structure to the tanker. This riser has an internal diameter of 12 inches and an outer diameter of 14.5 inches. As in the FPSO system, the PLEM for the FSO riser is located inside the mooring circle. The FSO system does not require a manifold system beneath the swivel.

The swivel required for the FPSO system has yet to be proven in an actual installation. The swivel details were developed from discussions with Single Buoy Moorings (SBM). They were advised of our production requirements. SMB indicated that a comparable system had been tested and was feasible. The cost estimate for the swivel was developed also based on discussions with SMB.

The swivel for the FSO system does not present any additional problems.

The FPSO subsea equipment was given a preliminary evaluation with the objective of sizing the members to withstand launch and drilling loads. The template is sized for eight wells with overall plan dimensions of 80 ft by 48 ft.

The FSO subsea equipment consists of only one PLEM, since all other equipment will be on the platform.

The deicing system is a major component of both the FPSO and the FSO systems due to the geographical location of the lease sale area. The icing problem will be more severe in the case of the FPSO since a large structure containing the facilities will be constructed on the tanker deck. This structure prevents icing of the process equipment which would otherwise be an even worse problem.

One of the biggest unknowns surrounding the icing problem is the ice accretion rate. For the lease sale area, an icing rate of 1.25 in./3 hrs. is the minimum design rate to be expected, when icing occurs. The maximum rate reported is about three times this, or 3.8 in./3 hrs. At these rates, the ice accumulation if allowed to accumulate could pose a threat to personnel, equipment, and possibly even the stability of the tanker itself.

The thermosyphon method requires approximately 1 KW/SQM to deice a vessel at a rate of 1.5 in./3 hrs. This translates to approximately 400,000 BTU/MIN for the FPSO and 60,000 BTU/min for the FSO.

The anticipated boilers on the tanker would have a capacity of approximately 1.3 million BTU/MIN. This is sufficient power to deice the vessel and run the other equipment even at the maximum ice rate and at the maximum production rate. Another alternative would be to heat the external surfaces electrically.

9.3 Costs and Schedules

The costs and schedules developed here for the FPSO and FSO systems were based on vendor supplied information to BWA in studies conducted over the past several years. One of the major areas of concern is the cost of the process equipment.

Previous studies, using several vendors have yielded costs for a 100,000 bopd system ranging from $20 million to $30 million. These figures include the process equipment as well as all the metering, piping, and safety equipment. The installation and hookup cost is also included in these figures.

It should be noted that certain items necessary for a production facility are included as part of the converted tanker. These items (quarters electrical, potable water, heating, fire fighting, etc.) yield a substantial savings when compared to constructing a completely new facility. Only costs for all modifications to these systems were included.

The costs for the FPSO system are presented in Table 9.3.1. Refer also to Appendix A. Two additional items were included in these costs; ice strengthening the tanker hull and a thruster assist system.

Due to the possibility of ice floes entering the FPSO area, the tanker hull will have to be strengthened. Assuming the thrusters can turn the tanker bow-on to the ice, most of the reinforcement will be concentrated near the bow. The strengthening of a 126,000 DWT vessel is estimated to require 12,300 short tons of steel. Using a cost of $1,200/ton installed in Japan, this translates to approximately 15 million dollars. The costs for purchase and installation of two 4,000 hp thrusters are approximately $1.2 million.

The specific hardware items included in Item 2 of Table 9.3.1 are listed in Tables 9.3.2 through 9.3.5. Note that the pipelines and control systems from the FLAB to the PLEM are costed separately.

The schedule for the FPSO system is shown in Figure 9.3.1.

The costs and schedules for the FSO system are similar to the FPSO except for the elimination of the process facilities and subsea equipment. As in the case of the FPSO, ice strengthening and thruster assist prices were included. The costs and schedule for the FSO are shown in Table 9.3.6 and Figure 9.3.2, respectively.

It should be noted that the costs associated with the purchase and/or lease of the required shuttle tankers have not been included in Tables 9.3.1 and 9.3.6. The costs for all modifications of the shuttle tankers have also been excluded.

9.4 Sensitivity to Critical Parameters

The various floating systems were evaluated for sensitivity to water depth and production rate. As much as possible, the major components of the system were held constant as to the size and number. For the 50,000 bopd case, the number of wells, templates, and risers were held constant. This gives a slightly conservative price estimate for these items.

For a reduction in production rate from 100,000 bopd to 50,000 bopd, the most siginificant price variation is in the cost of the process equipment. The cost of a 50,000 bopd facility is approximately 67% of the cost for a similar 100,000 bopd facility. All associated costs for secondary equipment, hook-up, and installation would similarily decrease.

The reduction in production rates from 100,000 bopd to 50,000 bopd has no affect on the details of the FSO system since the tanker size was held constant. The reduced rates would allow fewer loadings of shuttle tankers.

One of the most important parameters in the sensitivity study is the water depth. All previously discussed findings have been for a water depth of 250 ft. Mooring in a depth of 300 ft requires longer chains and risers, but no other significant changes are required. A depth of 150 ft was also studied. Using the same type of mooring system, i.e., eight leg, symmetric, all chain system, we were unable to design an acceptable system. The depth of 250 ft, which requires 5 3/4 inch chains, approaches the maximum capacity of present day mooring chains and is effectively the envelope of application of this mooring system type. The use of synthetic mooring components for extended periods as would be required in a production system yet has not been established. However, the application of these materials may allow operation in shallower depths.

Tables 9.4.1 and 9.4.2 show the resulting costs from the sensitivity study for the FPSO and FSO systems, respectively.

9.5 Seismic Effects on Floating Systems

The potential effects of seismicity in the form of seaquakes on floating systems was examined. A simplified one dimensional model was developed to assess the global behavior of a floating vessel under seaquakes and this is described in Figure 9.5.1. The key assumptions in the model are that seawater is incompressible and will not carry shear waves. Hence, the water is essentially a rigid column which will allow transmission of P waves to the surface. In essence, the horizontal motions at the surface are assumed to be zero in this simple model, because the water cannot transmit shear. The vertical motion is assumed to equal that of the mudline. Horizontal motions due to P waves striking the vessel at an oblique angle are ignored. Note that the motions developed are independent of water depth.

The maximum acceleration aboard the vessel as a function of damping is given in Figure 9.5.2. The full lateral design spectrum of Figure 2.3.4 was assumed to act in the vertical direction. The anticipated periods for all shuttle and production/storage tankers are in the 8 to 12 second range. For realistic heave dampings, the accelerations anticipated are less than 5% of gravity and significantly less than motions experienced under waves. While the high frequency motions would be disconcerting to the personnel aboard the vessel, creating an impression similar to that felt when running aground, seaquakes do not threaten the integrity of the global system.

While the global response of tankers in the 60,000 to 120,000 dwt range is moderate the potential local effects of seaquakes require further study. In particular, the pressures applied on local panels of tanker bottoms, out of the ice protection zone, need to be quantified. In addition, the possibility of local modes exciting various piping and equipment on the deck requires investigation. If seismic conditions should pose a problem in these areas, the costs of aleviating problems should not be severe. Proper equipment mounts incorporating damping should significantly improve performance without significant cost. Seaquakes could have a very severe impact, and may even threaten the safety of small shallow draft vessels such as support craft with a heave period of 0 to 2 seconds. This would place the vessels directly in the major energy band of the seaquake.

9.6 Conclusions and Recommendations

The overall conclusion from the information presented in the preceeding sections is that both the FPSO and FSO systems are feasible at least for water depths greater than 250 ft.

Specific conclusions of this study are:

- The FPSO and FSO are feasible in the moderate ice conditions experienced in the North Aleutian Basin. The mooring system can be designed to maintain station under ice and wave loads, provided that the tanker under thrust always meets significant ice pieces bow on. Limited ice strengthening was provided for the tanker to prevent local damage.

- An eight line, all chain mooring system provides acceptable capacity in water depths exceeding 250 ft. Below this a more extensive investigation is and required the feasibility of all chain system is not assured. Synthetic lines could be considered as an alternative.

- o A turret moored, flexible riser system (TMFR) provides the optimum solution for both FPSO and FSO cases. A total of 8 flexible risers were used connecting to flowlines and 8 subsea templates.

- o The North Aleutian Basin area has relatively high rates of ice accretion. Included in the design was a heating system to eliminate ice accumulation on the deck.

- o The piled steel jacket system and the concrete gravity platform considered in Chapter 5 and 7 are proven systems for the production rates and environmental conditons expected in the lease sale area. The floating production and storage systems have no experience at levels of production in the 100,000 bopd range in ice infested waters. Hence, the confidence levels in cost data and system efficiency are not as high as could be expected with the fixed platform concepts.

- o A series of cost and schedule tables and figures are presented in Section 9.3 for the FPSO and FSO systems. In comparing these costs with fixed platform costs the caution given in Section 4.0 should be noted.

10.0 REMOTE LOADING BUOY

The remote loading buoy concept was assumed in this study for cases where no floating vessels were available for storage. In two cases, the fixed structure was designed with integral storage. The location of the remote loading buoy with respect to the fixed structure was based on safety and cost. If the distance is too great a penalty would be paid in terms of pumping power required. If the distance is too short, a safety issue is raised concerning collision of the shuttle tanker with the fixed structure. Additional factors which govern the location of the remote loading buoy include: anticipated ice invasions, and water depth. The details of the remote loading buoy are discussed in the following sections.

10.1 Method of Approach

One of the major concerns in evaluating the remote loading buoy was the environmental loading when the shuttle tanker was not present. These environmental effects include: ice invasion, wave loads, and icing of the buoy itself.

Taking these effects into account, the counterweight articulated mooring, or CAM, was selected as the optimum concept for this study. The selection process was described in Chapter 3. The CAM system was designed to allow the shuttle tanker to remain on location in the 1 year storm. In the event of an ice invasion, the shuttle tanker would disengage and lower the CAM to the seabed. The shuttle tanker is then free to maneuver. It is assumed that the shuttle tankers would also require limited ice strengthening.

The swivel, mooring connector, and winching mechanism were all located on the shuttle tanker. The CAM, shown in Figure 3.5.1, is a symmetric, eight leg, all chain system, designed to develop the total restoring force. The dimensions of the CAM system were such that in the disconnected state, the top of the unit is 60 ft below the surface.

As in the case of the FSO system only one flexible riser was required. This riser would span from the PLEM to the base of the CAM.

10.2 Analysis Results and Evaluation

Using 3 inch diameter, grade 3 chain, a mean design static force of 166 kips was developed at an excursion of 21 ft. A computed shuttle tanker first order surge of 5 ft in the 1 year storm brings the total offset to 26 ft. At this offset, the tension in the most loaded line was 210 kips, 22% of the catalog breaking strength.

In this analysis, the effect of the buoyant column and counter weight have been ignored. Including these items would reduce the offset and the line tensions and hence, ignoring their effect was conservative. However, it is noted that the motion characteristics of the shuttle tankers have a significant effect on the system. (It is doubtful that the entire fleet of shuttle tankers would be identical.)

The flexible riser for the CAM system is the same as the riser used in the FSO system. This riser requires an internal diameter of 12 inches and a corresponding outer diameter of 14.5 inches.

For the 250 ft water depth the buoyant column used was 150 ft long with an outside diameter of 20 inches. This column was encased in foam, with an outside total diameter of 66 inches. For this configuration, a 10 kip counter weight was sufficient to pull the unit to the bottom when the shuttle tanker disconnects. The base of the counter weight was positioned 40 ft below the base of the buoyant column.

As in the case of the FPSO and FSO systems, the shuttle tanker would be vulnerable to ice accretion. An investigation into the requirements for deicing the shuttle tanker was briefly performed. The superstructure of the shuttle tanker used was larger than the superstructure of the captive tanker. The superstructure and the bow mounting for the CAM were the most important items requiring deicing with almost all of the energy required to prevent accumulation going to the superstructure. A detailed evaluation of the heating requirements for the shuttle tanker was not performed due to the variability of the superstructure size. Furthermore, the power generating capacity will vary from tanker to tanker. Also, it is noted that the shuttle tankers must be capable of navigating under their own power, unlike the FPSO and FSO vessels. Consequently, the shuttle tankers may require extra power generating capacity for deicing purposes.

10.3 Costs and Schedules

The cost for the mooring modifications to the bow of the shuttle tanker were based on vendor supplied information. These costs, as well as the costs for other components of the remote loading buoy are presented in Table 10.3.1. The costs for the shuttle tanker are not included in this study. Refer also to Appendix A.

The schedule for the remote loading buoy is shown in Figure 10.3.1.

10.4 Simulation of Loading System Efficiency and Downtime

A Monte Carlo simulation was carried out to establish the efficiency of the loading system and to estimate the downtime due to environmental conditions. Specifically included in the study were the following assumptions:

- Production or storage rate was either 50,000 bopd or 100,000 bopd and was nonvariable.

- The storage vessel had a capacity of 960,000 barrels.

- Production was shut down when wave conditions exceeded the one year storm H_S = 26 ft. After production was shut down it was assumed that 12 hours were required to restart production.

- The shuttle tanker had a capacity of 366,000 barrels and was loaded at 24,400 barrels per hour.

- Loading operations only occurred in seastates with H_S < 14 ft.

- Shuttle tanker approach and mooring was restricted to daylight conditions, visibilities greater than 2 miles and seastates with H_S < 8 ft.

- Allowances were made for required periods for paperwork loading preparations and preparations for sea subsequent to load. All shuttle tanker requirements are summarized in Table 10.4.1.

Specifically excluded from the study were the following considerations:

- Mechanical downtime and all downtime not directly associated with visibility, daylight conditions, and seastate.

- Ice incursions or any downtime associated with ice conditions. There is currently no available data to include the effects for this region. It is noted that ice effects will not be severe in the southern portion of the lease sale area, but ice will reduce efficiency in the Northern regions of the lease sale area.

The approach taken in the study was to develop a model for the environmental conditions. This model was then used to simulate a large number of design lives for the shuttle tanker system and the statistics of the number of hours of downtime were calculated from these results. The procedures are described in the following sections.

10.4.1 Simulation of Environmental Conditions

The principal environmental conditions to be simulated were the seastate conditions, visibility conditions and daylight/darkness occurrences. The basic time unit used in this study was a three hour period. The conditions were sampled during each three hour period and assumed to remain constant for the complete period. Each day consisted of 8 three hour periods and each year had 365.25 days, or 2,922 - 3 hour periods.

The prevailing seastate conditions were divided into 4 states as shown in Table 10.4.2. These states were chosen to conform to the various mooring, loading, and production envelopes previously described. A Markov model was used to model the seastate. Two years of wave data are available for the region. A typical trace is shown in Figure 10.4.1. The data was analyzed as shown in Figures 10.4.2 and 10.4.3. The probability of the wave conditions being in a specific wave state (Table 10.4.2) are shown in Figure 10.4.2. To simplify the simulation, the seastates were idealized using the simple model shown in Figure 10.4.3. The Markov transition matrix for the various states was evaluated for two seasons. April to September and October to March and the transition matrices are given in Tables 10.4.3 and 10.4.4.

Comparisons between the actual seastate conditions and the idealized seastates are given in Figures 10.4.4 and 10.4.5. The wave exceedance probabilities are shown to be in good agreement in Figure 10.4.4. The durations of the simulated wave states are shown to have excellent agreement on average with the actual seastate conditions in Figure 10.4.5. Perfect agreement would result in all the mean simulated points lying on the straight line. It was concluded that the simplified model shown for the seastate definition in Figure 10.4.4 and Tables 10.4.3 and 10.4.4 was an excellent representation of the actual wave conditions.

The daylight/darkness was represented for each 3 hour period as a function of month as shown in Figure 10.4.6. The availability of daylight ranges from a low of 2 periods (6 hours) in January to a high of 6 periods (18 hours) during the summer months.

The cutoff visibility was selected as 2 miles. The probability of having visibility less than 2 miles and the mean durations of this occurrence as a function of the month are given in Figures 10.4.7 and 10.4.8. Reduced visibility is very characteristic of the North Aleutian Basin. The data used in this study was collected at Port Moller. Reduced visibility is present for approximately one third of the time in summer and one fifth of the time in winter. The duration of these events is moderate ranging from a low of 4 hours on the average in October to almost 16 hours in July when reduced visibility occurrences reach their peak.

10.4.2 Simulation Results

The environmental conditions simulated in Section 10.4.1. were assembled into a Monte Carlo simulation of the production process. In the simulation each year was assumed to start at 12:00 a.m. on April 1. A total of 200 years of operation were simulated to generate the crude transport efficiency statistics.

The results of the simulation study for a number of different cases are given in Table 10.4.5. Results with an *, are shown for the base case. A number of sensitivity cases are also included.

Production rates of 50,000 and 100,000 bopd were studied. In the initial simulations, an unlimited supply of shuttle tankers was assumed. The permissible wave height at mooring was varied. With increasing production rate, an efficiency drop of almost 10% occurs when the allowable seastate is reduced from 8 to 6 ft. Reductions in storage capacity also cause significant efficiency drops at high production rates. The required shuttle tanker loading time is not as sensitive and has moderate impact on the production efficiency.

In the remaining simulations, the number of shuttle tankers was reduced. Two cases were considered. In the first case oil was transported to Seattle and in the second case to Valdez. At low production rates, for the distances covered in transit and for the number of tankers considered, the production efficiency was not affected by shuttle tanker numbers used. At higher production rates, there was a significant effect of the number of shuttle tankers on the production efficiency.

The simulation results indicate that production efficiency will be very good for the base cases considered. The overall efficiency will be reduced by mechanical downtime. Conservatively, the estimated downtime can be added to the present study results. In addition over much of the lease sale area, ice incursions may interrupt production and loading activities and load to reduced production efficiencies. At present there is no ice data available which gives the required joint probabilities of the duration, thicknesses, and areal coverage of the ice incursions, by time period. However, it is likely that ice will have an effect particularly in the Northern regions of the lease sale area every year and occurs in the winter months, which also include the most severe wave states. Hence, losses in efficiency due to ice should not be directly added to efficiency losses due to seastates. In general a complete simulation of both conditions is required.

10.5 Sensitivity to Critical Parameters

The variation in production rate does not affect the costing of the remote loading buoy. The production rate does affect the desired shuttle tanker schedule, but this does not significantly alter the costing of the system.

The variation in water depth, from 150-300 ft could have a significant impact on the overall feasibility of the system. In a specific case where the mooring chain configuration is held constant, and only the length of the buoyant column is changed, the behavior of the system would be virtually independent of water depth. This is because the mooring chains alone were used in the design to develop the required restoring force.

The variation of water depth changes the cost of the system slightly due to the changes in length of the buoyant column. The adjusted costs for the sensitivity study are given in Table 10.5.1.

The projected costs of modifying the shuttle tankers for appication with the remote loading buoy are included in Table 10.5.2.

The projected costs of modifying the shuttle tankers for application with the remote loading buoy are included in Table 10.5.2.

10.6 Conclusions and Recommendations

The CAM system described here was selected as the optimum configuration for a remote loading buoy for application in the North Aleutian Basin in combination with fixed platforms with integral storage. Specific conclusions for this study are:

- The system is feasible and can be designed for application in the water depth range evaluated in this study.
- A preliminary investigation into the effects of seaquakes on the FPSO and FSO systems indicate these systems to be safe from such occurances. However, hull bottom plate and framing, equipment mounts, etc., should be carefully designed.
- The system has the advantage of being removed from the ice and wave zones when shuttle tankers are not on station.
- The shuttle tanker and mooring system were designed for the tanker to remain on station during the one year storm.
- A simulation of the loading system efficiency indicates that the efficiency exceeds 95% for anticipated conditions in the region. No downtime was included for either ice incursions or mechanical problems.
- Modifications required to allow the shuttle tankers to moor to the CAM are feasible and relatively inexpensive. Power requirements may have to be increased on the shuttle tankers to allow for prevention of icing and this will require additional cost.
- Costs and schedule estimates were developed and are presented in Section 10.3.

11.0 PIPELINES

Pipelines are required in all scenarios considered and listed in Section 3.7. Pipelines were used in conjunction with the piled jacket platform in both scenarios where this system was considered. In the first field development scenario a combination marine and land pipeline was assumed for crude transport to a shore based terminal. In the second scenario where an FSO is used for storage, a one mile pipeline was assumed to connect the jacket to the riser loading the tanker.

In the case of both the hybrid steel/concrete and concrete gravity system, a pipeline was assumed to connect the integral storage to a remote loading station at a distance of one mile. Pipelines were used to connect the FPSO with each of eight subsea templates in the floating production option, leading to a requirement for 8 miles of pipeline.

The objectives of this preliminary study were to establish the feasibility and types of pipeline, methods of installation, and realistic cost estimates for the North Aleutian Basin. The primary reference used in deriving the information developed here was Reference 4.

11.1 Pipeline Design and Installation

Pipeline design is a combination of pipeline diameter, wall thickness, length, pumping capacity, end pressure and various additional requirements such as weight coating. The approach followed in this study was to use design and cost information from existing pipelines primarily in the North Sea, rather than attempt a more comprehensive design. In most cases API-5LX-42 pipe grade would be adequate but for long pipelines with high throughput API-5LX-52 pipe would be required. It was assumed that all pumping capacity was placed on the exporting platform and that 150% of the required pumping capacity was available. Intermediate booster pumps were not used because of excessive cost.

Two types of pipeline were in this study:

- o Tanker loading/unloading pipelines.
- o Long length marine pipelines delivery to shore or nearshore.

The required pipe diameter as a function of production rate can be estimated for both pipeline types from Figure 11.1.1 through 11.1.4. Figure 11.1.1 gives the required pipeline diameter for transport from the production platform to a floating storage tanker. Figure 11.1.2 gives the required pipeline diameter and installed horsepower for pipelines connecting the integral storage of the hybrid and concrete gravity platforms to the remote loading area and the shuttle tankers, as a function of tanker size. Figure 11.1.3 gives required diameters for long length marine pipelines, delivering crude to a shore based or nearshore terminal. For completeness and comparison, costs for a landbased pipeline are given in Figure 11.1.4.

The marine pipeline requires weight coating and water proofing to prevent corrosion. Typical weight coating requirements as a function of pipe diameters are given in Table 11.1.1.

Various options are available for marine pipeline installation. Six methods are illustrated in Figures 11.1.5 and 11.1.6. They include:

- o Conventional lay barge
- o Reel Method
- o Bottom tow method
- o Bottom pull method
- o Surface tow method
- o J-Pipelay

Taking into account the remote region, and the requirement to transport pipeline sections over long distances, the various bottom tow and pull and surface tow methods were not considered further. The reel method is limited to small diameter pipelines and taking into account the significant production rates this was excluded. Either the conventional lay barge or J-Pipelay methods appear to be the most attractive. The pipeline is assembled as it is installed. In developing cost estimates, the conventional lay barge method was assumed as this is likely to be the most readily available installation method for this region.

In the installation of the pipelines, while burial is not required (as it is in some ice infested regions), to prevent scour, etc., the additional protection afforded by burial of the pipeline was included in our cost estimate.

11.2 Cost Estimates

The cost estimate for marine pipelines was based on available cost data for the North Sea. To allow for the location of the North Aleutian Basin, a factor of 1.25 was applied to the North Sea costs. Included in the cost estimate were:

- Materials (pipe, coating, etc.)
- Transportation to site
- Mobilization/Demobilization of the required equipment
- Installation costs
- Pumping equipment cost computed at $2,500 per H.P. plus a cost of space allowance of $500 per H.P.

The estimated capital costs for the various pipelines used in this study are given in Figures 11.2.1 through 11.2.5. The cost of pipelines connecting production platforms to FSO's is given in Figure 11.2.1. For a production rate of 100,000 bopd and a distance of 1 mile, a cost of $MM12 is indicated.

For loading pipelines, the cost is given in Figure 11.2.2. For shuttle tankers of 60,000 DWT and a 1 mile pipeline the cost is approximately $MM23. Costs for long length marine and for completeness land based pipelines are given in Figures 11.2.3 through 11.2.5.

The cost estimates for the floating system included costs associated with installing central cables and were prepared separtely. The estimated cost for 16 miles of pipeline was $MM21.1. These are discussed in Appendix A.

11.3 Summary

A brief study of the various options for pipeline design and installation, and an estimate of the likely costs anticipated in the North Aleutian Basin was performed. It was concluded that all the required pipelines were feasible. Costs were anticipated to be approximately 25% higher than North Sea equivalents in preparing cost estimates.

12.0 FIELD DEVELOPMENT SCENARIOS

A number of potential field development scenarios are considered in this Chapter. The objective here is to provide typical applications of the data generated in Chapters 5, 6, 7, 9, 10, and 11. The following cases were considered:

Scenario 1: The production rate was taken as 100,000 bopd in 300 ft water depth. Production was by means of a jacket with a 170 mile, 32 inch diameter marine pipeline to shore. It was assumed that this pipeline was shared between 4 users. Specifically excluded in this scenario is all shore based development including the provision of land based pipelines. The estimated cost is given in Table 12.0.1.

Scenario 2: In this case, production was by means of a jacket in 300 ft water depth at 100,000 bopd. The production was sent by a 1 mile pipeline to a floating storage tanker. Specifically excluded from this scenario are all costs associated with the transport system using shuttle tankers. The estimated cost is given in Table 12.0.2.

Scenario 3: This is a repeat of scenario 1 with the water depth reduced to 150 ft and consequently the requirement for marine pipelines reduced to a 40 mile distance. The costs are given in Table 12.0.3.

Scenario 4: In this case the Hybrid concept is considered in conjunction with a remote loading buoy. The water depth is 300 ft and the production rate is 100,000 bopd. Specifically excluded from the costs are the shuttle tankers.

Scenario 5: This is a repeat of Scenario 4 in 150 ft water depth.

Scenario 6: In this case, for a water depth of 300 ft and a production rate of 100,000 bopd, the concrete gravity platform is used in conjunciton with a pipeline to a remote loading buoy. Again, specifically excluded from the cost are the transportation costs.

Scenario 7: This is a repeat of scenario 6 with the water depth reduced to 150 ft.

Scenario 8: In this case an FPSO is connected to 8 templates by two mile pipelines. All associated costs are included. Specifically excluded is the cost of shuttle tankers.

In this scenario, to ensure a complete comparison between the floating production system and the fixed drilling platforms the cost of providing drilling facilities was included. It was assumed that the drilling platform capital costs were equivalent to the purchase of two semisubmersibles for the 100,000 bopd case and one semisubmersible for the 50,000 bopd case. The drilling program may be completed by other means, but the capital cost of the facilities and platform have been assumed as stated.

The results are summarized for these 8 scenarios with production rates of 50,000 bopd and 100,000 bopd in Tables 12.0.9 and 12.0.10. In all cases Japanese fabrication was considered. For comparison, results where it was assumed that all fixed platforms were constructed on the U.S. West Coast are given in Table 12.0.11 for a production rate of 100,000 bopd.

The costs for fixed and floating system cannot be directly compared because of the different initial capital costs and ongoing leasing and operation costs implicit in both as summarized in Section 4.0.

13.0 GENERAL CONCLUSIONS

The objectives of this study were to assess the feasibility and costs of alternative production, storage and loading system for application in the North Aleutian Basin. Jackets, steel and concrete platforms with integral storage and floating tankers were considered as production systems. In addition, tankers were considered as storage systems. The type of remote loading system required for application with gravity platforms was recommended. Specific conclusions from this study as a whole are as follows:

- To meet the objectives of this study each concept was considered in a general way. The objective was to determine feasibility of the concepts together with major dimensions weights, costs and schedules. The analysis and design level reached in this study is compatable with that objective.

- The North Aleutian Basin has moderate ice conditions, wave conditions similar to design levels common in the Gulf of Mexico and severe seismic design criteria, which approach API Zone 5 design criteria. In addition the lease sale area has relatively high incidence of low visibility.

- Seismic loads control most of the member sizes on fixed platforms and the major dimensions of all fixed platforms. Reductions in seismic conditions produce significant savings in both material and fabrication costs. Seismic loads have little effect on floating systems.

- Ice loading controls the design of bracing crossing the ice region. These members must have increased wall thickness. In addition the conductor systems in all steel platform applications must be protected from ice by a protective cage around the conductors. All floating systems require local ice strengthening. For all systems the global effects of ice do not control.

- Wave loads do not play an important role in local member or global design for fixed platforms. Wave conditions do influence the mooring design and operating characteristics of floating systems.

- The fixed platform concepts designed here are based on tried and proven technology. The jacket system is double battered with a launch truss and is similar to the thousands of jackets currently in use around the world. The concrete gravity system is similar to concepts already in use in the North Sea. The Hybrid system is a combined jacket and concrete base, with some innovation required for the connection between them. The floating production and storage systems are conventional converted tankers with moderate ice strengthening. The TMFR system and the CAM system for loading have not been proven in practice, but they are based on technology that has. Hence, to develop feasible systems for use in the ice, seismic and wave conditions of the North Aleutian Basin requires no significant advances in current technology.

- Costs have been prepared for two fabrication sites:

  - Japan
  - U.S. West Coast

  The Japanese fabrication costs are lower. Transportation and towing costs are lower from the West Coast.

- Liquefaction under seismic conditions is a potential problem for the anticipated soil conditions and seismic conditions prevalent in the area. Piled jacket structures can be designed, at a cost of at least 20, to function under anticipated seismic and soil conditions. Gravity based systems are severely affected by potential liquefaction. Placement of gravity systems must be examined on a case by case basis and a detailed study of liquefaction potential for that specific site should be undertaken prior to significant design of the platform. Floating systems are relatively unaffected by liquefaction, although the position and capacity of the mooring systems must be reevaluated after significant seismic activity.

13.1 Specific Conclusions

Conclusions have been presented for each concept separately. The reader is referred to:

Section 5.5 - Piled Jackets
Section 6.5 - Hybrid Platforms
Section 7.6 - Concrete Platforms
Section 8.4 - Liquefaction Analysis
Section 9.6 - Floating Systems
Section 10.6 - Remote Loading Buoy
Section 12.0 - Typical Scenarios

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PRECIPITATION TYPE	FREQUENCY OF OCCURRENCE (%)
Rain/Drizzle	13.4
Freezing Rain	0.1
Snow/Sleet	11.3
Total	24.8

TABLE 2.1.1 FREQUENCY OF OCCURRENCE OF PRECIPITATION TYPES

MONTH	MEAN WIND (KTS)	% > 34 KTS
Jan	18	8
Feb	19	8
Mar	16	4
April	16	4
May	13	1
June	11	1
July	11	1
Aug	14	2
Sept	17	4
Oct	19	7
Nov	20	12
Dec	18	7

TABLE 2.2.1 ANNUAL WIND CONDITIONS

RETURN PERIOD (YR)	1 MIN. AVG. WIND (KTS)
100	106
50	98
25	90
10	81
1	66
1 Month	57

TABLE 2.2.2 DESIGN WIND CONDITIONS

RETURN PERIOD (YR)	H_s (FT)	H_{max} (FT)	T_p (SEC)
100	38	71	14
50	36	67	13
25	34	63	12
10	32	60	11
1	26	48	10.5
1 Month (Summer)	14	26	8.0
1 Month (Winter)	26	48	10.5

* JONSWAP spectrum used.

TABLE 2.2.3 EXTREME WAVE CONDITIONS

RETURN PERIOD (YRS)	EXTREME SURFACE CURRENT (KTS)
100	2.0
50	1.5
25	1.2
10	1.0
1	0.9
Monthly	0.5

TABLE 2.2.4 EXTREME SURFACE CURRENTS AS A FUNCTION OF RETURN PERIODS

RETURN PERIOD (YR)	SUSTAINED WIND (KT)	H_s (FT)	H_{max} (FT)	T_p (SEC)	CURRENT (KT)	SURGE + TIDE (FT)
100	106	38	71	14.0	2.0	8
1	66	26	48	10.5	0.9	6
1 Month	57	26 (winter)	48	10.5	0.5	5
		14 (summer)	26	8.0		

Design Wave Spectrum Taken as JONSWAP

TABLE 2.2.5 OCEANOGRAPHIC CRITERIA SUMMARY

AREA	AVERAGE ANNUAL ICE GROWTH (INCH)
Nunivak Island	36
Nakner	33
Pribilof Islands	18

TABLE 2.4.1 EXPECTED ICE GROWTHS FOR AREAS CONTIGUOUS TO THE LEASE SALE AREA

AREA	SHEET ICE THICKNESS (INCH)	RAFTED ICE THICKNESS (INCH)
North Aleutian Basin (proposed)	24	48
St. George Basin	28	55
Navarin Basin	39	79
Cook Inlet	25-30	42-60

TABLE 2.4.2 A COMPARISON OF THE PROPOSED SHEET ICE PROPERTIES IN THE NORTH ALEUTIAN BASIN WITH EXISTING STRUCTURE DESIGN THICKNESS

STRUCTURE	LEG DIAMETER (FT)	LOAD (kips) DESIGNED	SUGGESTED
MGS "A"	14.5	2,830	2,000
MGS "C"	15.5	2,340	2,140
MONOPOD	28.5	7,410	3,933

TABLE 2.4.3 A COMPARISON OF THE AS DESIGNED LOADS FOR STRUCTURES IN COOK INLET, AND COMPUTED LOADS FOR THE CRITERIA DEVELOPED FOR THE NORTH ALEUTIAN BASIN

	CASE I	CASE II
Rate (BOPD)	50,000	100,000
Depth TVD (ft)	10,000	10,000
Producer Well Rate (BOPD)	2,000	2,500
No. of Producers	25	40
No. of Injection Wells	5	8
Spare Slots	6	8
Total Slots	36	56
Oil Gravity (API°)	35	35
Sulphur	0.1-0.2	0.1-0.2
Oil Temp. at Surface (°F)	200	200
Reservoir Pressure (psi)	5,000	5,000
Pressure Gradient (psi/ft)	0.7	0.7
Inlet separation pressure (psi)	200	200

TABLE 2.6.1 OIL PRODUCTION CRITERIA

	CASE I	CASE II
GOR (SCF/BBL)	1,500	1,500
Total Gas Rate (MMCFD)	75	150
Water Vapor (LBS/MMCF)	0.5	0.5
Gas Gravity	0.85	0.85
Special Problems	None	None

TABLE 2.6.2 GAS PRODUCTION CRITERIA

	CASE I	CASE II
Water Cut (%)	50	50
Injection Rate/Well (BWPD)	2,000	2,500
Injection Rate (Total) (BWPD)	50,000	100,000
Injection Pressure (psi)	5,500	5,500
No. of Injection Wells	(included in oil production)	

Other

No. of Rigs	1	2
Combined Drilling/Production	Yes	Yes

TABLE 2.6.3 WATER AND OTHER CRITERIA

ITEM	CASE I - 50,000 BOPD AREA (ft²)	DL (kips)	DL + LL (kips)	CASE II - 100,000 BOPD AREA (ft²)	DL (kips)	DL + LL (kips)
Deck Structure	-	5,000	5,000	-	5,000	5,000
Oil Facilities	18,500	4,500	7,000	22,500	5,500	8,300
Gas Handling	7,500	3,500	4,500	10,500	5,000	6,300
Water/Flood	4,000	1,200	2,500	6,000	2,000	4,500
Generators/Electrical	9,000	4,000	4,100	13,000	4,500	4,600
Utilities	4,500	1,000	2,500	-	1,500	3,000
Quarters (Max Crew Size)	9,000	1,200 (125)	1,200	10,000	1,500 (140)	1,500
Miscellaneous	-	1,500	1,800	-	2,000	2,500
Drilling Equipment	26,000	5,500	17,000	26,000	6,000	24,300
Total	78,500	27,400 kips	45,600 kips	94,000	33,000 kips	60,000 kips
No. of Well Slots		36			56	
No. of Rigs		1			2	

Module steelwork included in weights. 2,000 tons for 50,000 BOPD, 3,000 tons for 100,000 BOPD cases.

TABLE 2.6.4 FACILITIES WEIGHT AND REQUIRED AREA DATA

LOCATION	1/1,000 HEAVE (FT)	1/1,000 PITCH (DEG)
Tanker at C.G.	11.3	4.7
Tanker at Turret	27.3	4.7

TABLE 3.4.1 CAPTIVE TANKER MOTIONS FOR 100 YEAR STORM

SOURCE	FORCE (KIPS)
100 Year Wave	71.7
100 Year Current	83.6
100 Year Wind	214.3
Total 1 + 2 + 3	369.6
2 ft Ice Thickness	78.0
4 ft Ice Thickness	231.0
6 ft Ice Thickness	458.5

* Ice loads act only on tanker bow.

TABLE 3.4.2 STEADY LOADS ON CAPTIVE TANKER

BUOYS

RIGID RISER BUOY	
Riser Weight	84 S. Tons
Riser Tension	50 S. Tons
Mooring Chains	188 S. Tons
Swivel	110 S. Tons
Yoke	175 S. Tons
Bouy	50 S. Tons
	657 S. Tons
30 Ft Diameter	40 Ft Draft

BUOY FORCES	
Current	3.4 S. Tons
Wave Drift	3.9 S. Tons
Riser Current	0.6 S. Tons
	7.9 S. Tons
Dynamic Pressure Variation	428 S. Tons
(Vertical Force Due to Wave on Restrained Buoy)	

TABLE 3.4.3 BUOY WEIGHTS AND FORCES FOR RIGID RISERS IN THE SALM CONCEPT

FLEXIBLE RISER BUOY	
Riser Weight	54 S. Tons
Mooring Chains	188 S. Tons
Swivel	110 S. Tons
Yoke	175 S. Tons
Buoy	50 S. Tons
	577 S. Tons

BUOY FORCES		
Current	2.5 S. Tons	
Wave Drift	1.4 S. Tons	
Riser Current	1.0 S. Tons	(each x 10)
	13.9 S. Tons	

TABLE 3.4.4 BUOY WEIGHTS AND FORCES FOR FLEXIBLE RISERS IN THE CALM CONCEPT

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Usuage of Proven Concepts	6	9	54	5	30	7	42	6	36	7	42
Usuage of Proven Design Techniques	9	10	90	9	81	10	90	9	81	10	90
Ease of Fabrication	4	8	32	7	28	10	40	10	40	10	40
Ease of Transportation	4	10	40	9	36	8	32	7	28	8	32
Ease of Installation	4	10	40	8	32	9	36	8	32	10	40
Weather Vaning Ability	6	7	42	7	42	10	60	10	60	10	60
Mooring Motions	6	10	60	5	30	7	42	7	42	6	36
Mooring Loads	7	10	70	9	63	6	42	6	42	7	49
Moor/Riser Interference	4	7	28	10	40	10	40	10	40	7	28
Total Score (Max. 500)			456		382		424		401		417
Percentage			0.91		0.76		0.85		0.80		0.83

* S is score out of 10

TABLE 3.4.5 TECHNICAL MERIT CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Loss of Deck Area	10	5	50	5	50	10	100	10	100	10	100
Loss of Storage Vol.	9	5	45	5	45	10	90	10	90	10	90
Required Modifications	10	5	50	5	50	8	80	8	80	8	80
Required Strengthening	7	7	49	6	42	7	49	7	49	7	49
Use of Existing Facil.	10	8	80	8	80	9	90	9	90	9	90
Specialization of Vessel	4	6	24	6	24	8	32	8	32	8	32
Total Score (Max. 500)			298		291		441		441		441
			0.60		0.58		0.88		0.88		0.88

* S is score out of 10.

TABLE 3.4.6 IMPACT ON CAPTIVE TANKER CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Hull Modifications	15	5	75	5	75	9	135	9	135	9	135
Hull Strengthening	10	8	80	7	70	8	80	8	80	8	80
Hull Ice Protection	3	9	27	9	27	9	27	9	27	9	27
Equip. Ice Protection	7	9	63	9	63	7	49	7	49	7	49
Superstructures	10	10	100	5	50	10	100	10	100	10	100
Outfit	5	10	50	7	35	10	50	10	50	10	50
Total Score (Max. 500)			395		320		441		441		441
Percentage			0.79		0.64		0.88		0.88		0.88

*S is score out of 10.

TABLE 3.4.7 HULL AND OUTFIT COST CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Mooring Leg(s)	9	10	90	10	90	8	72	7	63	9	81
Mooring Vessel Conn.	6	10	60	10	60	7	42	7	42	7	42
Mooring Seabed Conn.	4	9	36	9	36	8	32	6	24	9	36
Mooring Bearings	5	7	35	7	35	9	45	9	45	9	45
Mooring Articulations	3	10	30	10	30	8	24	8	24	10	30
Mooring Ice Protect.	4	10	40	10	40	7	28	7	28	6	24
Riser Body	8	9	72	5	40	10	80	10	80	8	64
Riser Support Fac.	4	9	36	4	16	8	32	8	32	7	28
Swivels, Jumper Pipes	4	9	36	5	20	8	32	8	32	8	32
Riser Ice Protect.	3	10	30	10	30	8	24	8	24	7	21
Total Score (Max. 500)			465		397		411		394		403
Percentage			0.93		0.79		0.82		0.79		0.81

*S is score out of 10.

TABLE 3.4.8 MOORING AND RISER COST CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Required Maint.	12	10	120	9	108	8	96	8	96	8	96
Expected Repairs	8	10	80	9	72	7	56	7	56	7	56
Equipm. w/Special Oversize Parts	5	10	50	8	40	10	50	10	50	10	50
Inventory	7	10	70	9	63	9	63	9	63	9	63
Overhead	6	10	60	10	60	10	60	10	60	10	60
Crew	8	10	80	9	72	9	72	9	72	9	72
Ease of Inspection	4	9	36	9	36	8	32	8	32	8	32
Total Score (Max. 500)			496		451		429		429		429
Percentage			0.99		0.90		0.86		0.86		0.86

*S is score out of 10.

TABLE 3.4.9 OPERATING CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Suitability for Arctic	6	9	54	9	54	7	42	7	42	6	36
Inherent Reliability	9	10	90	9	81	8	72	8	72	7	63
Ice Protection	6	10	60	10	60	5	30	5	30	5	30
Wave Protection	6	10	60	9	54	6	36	6	36	6	36
Emerg. Disconn. Capab.	4	9	36	7	28	5	20	5	20	5	20
Safety of Equip.	8	9	72	9	72	7	56	7	56	6	48
Safety of Crew	11	10	110	10	110	9	99	9	99	9	99
Total Score (Max. 500)			482		459		355		355		332
Percentage			0.96		0.92		0.71		0.71		0.66

* S is score out of 10.

TABLE 3.4.10 RELIABILITY AND SAFETY CATEGORY

PARAMETER	WEIGHT W	TURRET MOORED FLEXIBLE RISER		TURRET MOORED RIGID RISER		SALM		CAT		CALM	
		SCORE									
		S*	WxS	S	WxS	S	WxS	S	WxS	S	WxS
Technical Merit	.40	456	182	382	153	424	170	401	160	417	167
Impact on Captive Tanker	.25	298	75	291	73	441	110	441	110	441	110
Hull and Outfit Cost	.35	395	138	320	112	441	154	441	154	441	154
Mooring & Riser Cost	.35	465	163	397	139	411	144	394	138	403	141
Operating Cost	.30	496	149	451	135	429	129	429	129	429	129
Reliability & Safety	.35	482	169	459	161	355	124	355	124	332	116
Total Score (Max. 1000)			876		773		831		815		817
Rank			1		5		2		4		3

*S is score out of 500 from Tables 3.4.5 to 3.4.10.

TABLE 3.4.11 RANKING SUMMARY FOR FLOATING SYSTEMS

LOCATION	1/1000 HEAVE (FT)	1/1000 PITCH (DEG)
C.G.	4.6	2.8
BOW	21.3	2.8

TABLE 3.5.1 EXTREME SHUTTLE TANKER MOTIONS IN 1 YEAR STORM

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Mooring Loads	8	8	64	6	48	6	48	8	64
Mooring System Motions	8	8	64	4	32	6	48	8	64
Weather Vaning Ability	8	10	80	10	80	10	80	10	80
Mooring/Riser Interference	8	8	64	6	48	10	80	10	80
Usage of Untried Concepts	7	6	42	6	42	6	42	6	42
Ease of Transp.	4	8	32	10	40	6	24	8	32
Ease of Inspec.	4	10	40	6	24	6	24	6	24
Riser Load Variation	3	10	30	10	30	2	6	8	24
Total Score (Max. 500)			416		344		352		410
Percentage			0.83		0.69		0.70		0.82

* S is score out of 10.

TABLE 3.5.2 TECHNICAL MERIT CATEGORY

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Usage of Special Materials	9	10	90	6	54	6	54	6	54
Ice Protection	26	10	260	4	104	4	104	2	52
Hull Modifications	15	6	90	10	150	10	150	10	150
Total Score (Max. 500)			440		308		308		256
Percentage			0.88		0.62		0.62		0.51

* S is score out of 10.

TABLE 3.5.3 HULL AND OUTFIT COSTS CATEGORY

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Mooring System Cost	22	6	132	10	220	8	176	6	132
Riser Equipment Cost	21	8	168	6	126	8	168	8	168
Ease of Fabrication	7	6	42	10	70	8	56	8	56
Total Score (Max. 500)			342		416		400		356
Percentage			0.68		0.83		0.80		0.71

* S is score out of 10.

TABLE 3.5.4 MOORING AND RISER COSTS CATEGORY

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Maintenance and Repair Costs	30	10	300	4	120	4	120	4	120
Ease of Installation	8	6	48	10	80	6	48	4	32
Ease of Shuttle Connection	12	8	96	6	72	6	72	6	72
Total Score (Max. 500)			444		272		240		224
Percentage			0.89		0.54		0.48		0.45

* S is score out of 10.

TABLE 3.5.5 OPERATING COSTS CATEGORY

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Overall Reliability and Safety	24	6	144	6	144	8	192	6	144
Sea Protection	10	10	100	6	60	6	60	8	80
Suitability for Arctic	10	10	100	4	40	4	40	4	40
Emergency Disconn.	6	8	48	10	60	10	60	10	60
Total Score (Max. 500)			392		304		352		344
Percentage			0.78		0.61		0.70		0.69

* S is score out of 10.

TABLE 3.5.6 RELIABILITY AND SAFETY CATEGORY

PARAMETER	WEIGHT W	CAM (FLEX RISER)		CALM (FLEX RISER)		SALM (RIGID RISER)		CAT (RIGID RISER)	
		SCORE							
		S*	WxS	S	WxS	S	WxS	S	WxS
Technical Merit	.46	416	191	344	158	352	162	410	189
Hull and Outfit Cost	.40	440	176	308	123	308	123	256	102
Mooring and Riser	.40	342	137	416	166	400	160	356	142
Operating	.34	444	151	272	92	240	82	224	76
Reliability and Safety	.40	392	157	304	122	352	141	344	138
Total Score (Max. 1000)			812		661		668		647
Rank			1		3		2		4

* S is score out of 500 from Tables 3.5.2 to 3.5.6.

TABLE 3.5.7 RANKING SUMMARY

BASE CASE

Deck Weight	60,000 kips
Water Depth	300 ft
Soil Springs	
- Translation	2×10^5 kips/ft
- Rotation	2×10^{10} kip ft/radian

FROM	RANGE	TO
1×10^5	Translation Spring	4×10^5
1×10^{10}	Rotational Spring	4×10^{10}
150 ft	Water Depth	300 ft
40,000 kips	Deck Weight	100,000 kips

TABLE 5.1.1 BASE CASE AND PARAMETER SENSITIVITY FOR SIMPLIFIED MODEL

MODE	PERIOD (sec)
1	1.82
2	1.81
3	1.34
4	0.57
5	0.56
6	0.44
7	0.41
8	0.33
9	0.33
10	0.28

TABLE 5.2.1 MODAL PERIODS FOR 300 FT WATER DEPTH

COMBINATION			RMS VALUE BASE SHEAR (KIPS)	OTM (KIP.INCH)
X	Y	Z		
1	2/3	1/2	38,075	1.34×10^8
2/3	1	1/2	37,887	1.34×10^8
1	0	0	31,710	1.11×10^8
0	1	0	31,300	1.11×10^8
0	0	1	4,187	0.10×10^8

TABLE 5.2.2 RMS BASE SHEAR AND OVERTURNING MOMENTS FOR VARIOUS DIRECTIONAL COMBINATIONS

MODE	SHEAR (KIPS)	MOMENT (KIP INCH)
1	28,084	11.06×10^7
2	19,190	7.44×10^7
3	33	0.02×10^7
4	8,145	0.54×10^7
5	14,610	0.85×10^7
6	1,417	0.47×10^7
7	366	0.04×10^7
8	1,196	0.51×10^7
9	2,253	0.25×10^7
10	2,076	0.34×10^7

TABLE 5.2.3 SHEAR AND MOMENT IN THE INDIVIDUAL MODES

CONDITION	BASE SHEAR	AXIAL LOADS CORNER PILE	AXIAL LOADS INTERIOR PILE
Seismic (1:2/3:1/2)	37.3	4.5	3.3
Seismic (2/3:1:1/2)	37.1	4.0	4.2
Dead Only (Typical)	-	2.6	3.4
Dead and Wave	3.5	2.9	3.5

TABLE 5.2.4 SUMMARY OF AXIAL LOADS IN THOUSANDS OF KIPS IN THE PILES

		H (FT)	T (SEC)	DIRECTION	CASE A SHEAR	CASE A MOMENT	CASE B SHEAR	CASE B MOMENT
1	X	71	14	0°	3,511	.946	7,248	1.643
2	X	71	15	45°	2,554	-.565	5,214	1.227
	Y				2,319	.226	5,116	-1.177
3	Y	71	14	90°	3,943	.242	7,378	-1.694
1	Y	71	10	90	4,245	-1.130	9,475	-2.282
2	Y	71	12	90	3,896	-.997	8,134	-1.895
3	Y	71	14	90	3,511	-.946	7,378	-1.694
4	Y	71	16	90	4,189	-.949	6,750	-1.489
5	Y	71	18	90	4,438	-.953	6,180	-1.341

FORCE = KIPS

MOMENT = ($x10^6$ KIP-FT)

Case A: Equivalent diameter of conductors based on volume of conductors.

Case B: Conductors enclosed in 2-40 ft diameter cylinders.

X Wave directed on Rows 1 or 4.

Y Wave directed on Rows A or B.

TABLE 5.2.5 SUMMARY OF WAVE LOADS - 300 FT WATER DEPTH

MEMBER	EXPOSED WIDTH (in)	LOADED AREA (ft^2)	CURVE PRESSURE (psi)	DESIGN PRESSURE (psi)
Legs	72	36	230	230
Bracing:				
End-Vertical	36	18	345	345
End-Diagonal	78	39	220	220
Side-Ends	66	33	235	235
Side-Center	90	45	205	205
Ice Cage	36	18	345	85

TABLE 5.2.6 DESIGN ICE PRESSURES FOR A 6 FT ICE THICKNESS ON EXPOSED PILED JACKET MEMBERS

MODE	PERIOD (SEC)
1	1.24
2	1.11
3	0.96
4	0.37
5	0.35
6	0.34
7	0.28
8	0.28
9	0.25
10	0.24

TABLE 5.2.7 PERIODS FOR 150 FT DEPTH SINGLE PIECE JACKET

COMBINATION			SHEAR	MOMENT
X	Y	Z	(KIPS)	(KIP.INCH)
1	2/3	1/2	47,525	11.06×10^7
2/3	1	1/2	49,726	11.47×10^7
1	0	0	37,858	8.89×10^7
0	1	0	42,682	9.80×10^7

X Direction along Rows A and B

Y Direction along Rows 1 and 4.

Z Direction Vertical

TABLE 5.2.8 BASE SHEARS AND MOMENTS FOR 150 FT WATER DEPTH, SINGLE PIECE JACKET

MODE	PERIOD
1	1.30
2	1.06
3	0.87 TORSION
4	0.41
5	0.35
6	0.32
7	0.26
8	0.19
9	0.19
10	0.19

TABLE 5.2.9 FUNDAMENTAL PERIODS FOR THE MULTIPLE PLATFORM, 150 FT WATER DEPTH CASE

COMBINATION			SHEAR	MOMENT
X	Y	Z	(KIPS)	(KIP.INCH)
1	2/3	1/2	14,882	3.3×10^7
2/3	1	1/2	13,699	3.0×10^7
1	0	0	13,103	2.9×10^7
0	1	0	10,505	2.3×10^7

X Direction along Rows A and B

Y Direction along Rows 1 and 4

Z Direction Vertical

TABLE 5.2.10 BASE SHEARS AND MOMENTS FOR MULTIPLE PLATFORM IN 150 FT WATER DEPTH CASE

	ITEM	SUB-TOTAL (kips)	TOTAL (kips)
1	Basic Steel Structure		
	: Main Framing*	14,100	
	: Ice Cage	700	
	: Conductor Guide Framing	1,700	
	: Skirt Guide/Sleeves	1,600	
	: Joint Cans	1,400	19,500
2	Buoyancy Tank/Bracing	-	-
3	Mud-Mats		
	: Steel	400	
	: Timber	200	600
4	Corrosion		
	: Allowance	100	
	: Anodes	600	700
5	Miscellaneous		500
6	Sub-Total		21,300
7	Piles		
	: Main	6,700	
	: Skirt	13,000	19,700
8	Conductors (60)		6,900
9	Total		47,900

* To +70 Ft.

TABLE 5.2.11 WEIGHT BREAKDOWN - 300 FT WATER

	ITEM	SUB-TOTAL (kips)	TOTAL (kips)
1	Basic Steel Structure		
	: Main Framing	10,800	
	: Ice Cage	700	
	: Conductor Guide Framing	1,200	
	: Skirt Guide/Sleeves	1,600	
	: Joint Cans	1,100	15,400
2	Buoyancy Tank/Bracing	-	-
3	Mud-Mats		
	: Steel	400	
	: Timber	200	600
4	Corrosion		
	: Allowance	100	
	: Anodes	600	700
5	Miscellaneous		500
6	Sub-Total		17,200
7	Piles		
	: Main	5,200	
	: Skirt	13,000	18,200
8	Conductors (60)		4,100
9	Total		39,500

TABLE 5.2.12 WEIGHT BREAKDOWN - 150 FT WATER

	ITEM	SUB-TOTAL (kips)	TOTAL (kips)
1	Basic Steel Structure		
	: Main Framing	4,860	
	: Ice Cage	700	
	: Conductor Guide Framing	380	
	: Skirt Guide/Sleeves (In Main Frame)	50	
	: Joint Cans	500	6,490
2	Buoyancy Tank/Bracing		300
3	Mud-Mats		
	: Steel	100	
	: Timber	100	200
4	Corrosion		
	: Allowance	30	
	: Anodes	1500	180
5	Miscellaneous		130
6	Sub-Total		7,300
7	Piles		
	: Main (4 @ 653)	2,610	
	: Skirt (8 @ 555)	4,440	7,050
8	Conductors (30)		2,050
9	Total		16,400

TABLE 5.2.13 $16{,}000^{K}$ DECK LOAD - DRILLING PLATFORM, WEIGHT BREAKDOWN - 150 FT WATER

ITEM		SUB-TOTAL (kips)	TOTAL (kips)
1	Basic Steel Structure		
	: Main Framing	4,860	
	: Ice Cage	-	
	: Conductor Guide Framing	-	
	: Skirt Guide/Sleeves (In Main Frame)	50	
	: Joint Cans	500	5,410
2	Buoyancy Tank/Bracing		220
3	Mud-Mats		
	: Steel	100	
	: Timber	100	200
4	Corrosion		
	: Allowance	30	
	: Anodes	150	180
5	Miscellaneous		130
6	Sub-Total		6,140
7	Piles		
	: Main	2,610	
	: Skirt	4,440	7,050
9	Total		13,190

TABLE 5.2.14 16,000^{K} DECK LOAD - PRODUCTION OR ACCOMODATION OR UTILITIES STRUCTURE, WEIGHT BREAKDOWN - 150 FT WATER

			($MM)
1.	Engineering, Management (10%)		
	Marine Insurance		23.3
2.	Jacket:		
	Materials	5.9	
	Fabrication	13.7	
	Transportation	2.5	
	Installation	4.7	26.8
3.	Deck:		
	Materials	1.5	
	Fabrication	4.5	
	Transportation	0.6	
	Installation	0.3	6.9
4.	Piles:		
	Materials	8.0	
	Fabrication	2.7	
	Transportation	2.0	
	Installation	20.3	33.0
5.	Facilities:		
	Steel and Fabrication	8.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Accommodation Module	24.0	
	Yard Assembly	30.8	
	Transport	5.2	
	Installation	1.1	
	Offshore	8.5	166.0
6.	Total		256.0

TABLE 5.3.1 ESTIMATED COST, PILED STEEL JACKET, 300 FT WATER DEPTH, JAPAN, 100,000 BOPD PRODUCTION

			($MM)
1.	Engineering, Management (10%) Marine Insurance		27.5
2.	Jacket:		
	Materials	5.9	
	Fabrication	16.7	
	Transportation	1.3	
	Installation	4.7	28.6
3.	Deck:		
	Materials	1.5	
	Fabrication	5.3	
	Transportation	0.4	
	Installation	0.3	7.5
4.	Piles:		
	Materials	8.0	
	Fabrication	5.3	
	Transportation	1.7	
	Installation	20.3	35.3
5.	Facilities:		
	Steel and Fabrication	9.0	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Accommodation Module	27.0	
	Yard Assembly	65.8	
	Transport	3.7	
	Installation	1.1	
	Offshore Hookup/Commissioning	8.5	203.1
6.	Total		302.0

TABLE 5.3.2 ESTIMATED COST, PILED STEEL JACKET, 300 FT WATER DEPTH, U.S. WEST COAST, 100,000 BOPD PRODUCTION

		($MM)	
1.	Engineering, Management Marine Insurance (10%)		22.4
2.	Jacket:		
	Materials	4.6	
	Fabrication	10.8	
	Transportation	2.5	
	Installation	4.2	22.1
3.	Deck:		
	Materials	1.5	
	Fabrication	4.6	
	Transportation	0.6	
	Installation	0.3	7.0
4.	Piles:		
	Materials	6.7	
	Fabrication	2.2	
	Transportation	1.9	
	Installation	17.7	28.5
5.	Facilities:		
	Steel and Fabrication	8.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Accommodation Module	24.0	
	Yard Assembly	30.8	
	Transport	5.2	
	Installation	1.1	
	Offshore Hookup	8.5	166.0
6.	Total		246.0

TABLE 5.3.3 ESTIMATED COST, PILED STEEL JACKET, 150 FT WATER DEPTH, JAPAN, 100,000 BOPD PRODUCTION

		($MM)	
1.	Engineering, Management Marine Insurance (10%)		26.5
2.	Jacket:		
	Materials	4.6	
	Fabrication	13.1	
	Transportation	1.3	
	Installation	4.2	23.2
3.	Deck:		
	Materials	1.5	
	Fabrication	5.4	
	Transportation	0.4	
	Installation	0.3	7.6
4.	Piles:		
	Materials	6.7	
	Fabrication	4.7	
	Transportation	1.6	
	Installation	17.7	30.7
5.	Facilities:		
	Steel and Fabrication	9.0	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Accommodation Module	27.0	
	Yard Assembly	65.8	
	Transport	3.7	
	Installation	1.1	
	Offshore Hookup/Commissioning	8.5	203.1
6.	Total		291.1

TABLE 5.3.4 ESTIMATED COST, PILED STEEL JACKET, 150 FT WATER DEPTH, U.S. WEST COAST, 100,000 BOPD PRODUCTION

		($MM)
1. Engineering, Management Marine Insurance (10%)		3.5
2. Jacket:		
Materials	2.2	
Fabrication	5.1	
Transportation	1.6	
Installation	2.9	11.8
3. Deck:		
Materials	0.8	
Fabrication	2.3	
Transportation	0.6	
Installation	0.3	4.0
4. Piles:		
Materials	2.7	
Fabrication	0.9	
Transportation	1.2	
Installation	14.1	18.9
5. Total		38.2

*No facilities costs included.

TABLE 5.3.5 ESTIMATED COST, MULTIPLE JACKET, 150 FT WATER DEPTH, JAPAN

		($MM)
1. Engineering, Management Marine Insurance (10%)		3.6
2. Jacket:		
Materials	2.2	
Fabrication	6.2	
Transportation	0.8	
Installation	2.9	12.1
3. Deck:		
Materials	0.8	
Fabrication	2.6	
Transportation	0.4	
Installation	0.3	4.1
4. Piles:		
Materials	2.7	
Fabrication	1.8	
Transportation	1.1	
Installation	14.1	19.7
5. Total		39.5

* No facilities costs included.

TABLE 5.3.6 ESTIMATED COSTS MULTIPLE JACKET, 150 FT WATER DEPTH, U.S. WEST COAST

			($MM)	
1.	Engineering, Management Marine Insurance (10%)			30.3
2.	Jacket:			
	Materials	8.8		
	Fabrication	20.4		
	Transportation	6.4		
	Installation	11.6		47.2
3.	Deck:			
	Materials	3.2		
	Fabrication	9.2		
	Transportation	2.4		
	Installation	1.2		16.0
4.	Piles:			
	Materials	9.6		
	Fabrication	3.2		
	Transportation	4.8		
	Installation	56.4		74.0
5.	Facilities			
	Steel and Fabrication	8.4		
	Drilling Facilities	34.0		
	Production Facilities	54.0		
	Accommodation Module	24.0		
	Yard Assembly	30.8		
	Transport	5.2		
	Installation	1.1		
	Offshore Hookup/Commissioning	8.5		166.0
6.	Total			333.5

TABLE 5.3.7 ESTIMATED COSTS, 4 JACKETS, 150 FT WATER DEPTH, JAPAN

		($MM)
1. Engineering, Management Marine Insurance (10%)		34.5
2. Jacket:		
Materials	8.8	
Fabrication	24.8	
Transportation	3.2	
Installation	11.6	48.4
3. Deck:		
Materials	3.2	
Fabrication	10.4	
Transportation	1.6	
Installation	1.2	16.4
4. Piles:		
Materials	9.6	
Fabrication	6.4	
Transportation	4.4	
Installation	56.4	76.8
5. Facilities		
Steel and Fabrication	9.0	
Drilling Facilities	34.0	
Production Facilities	54.0	
Accommodation Module	27.0	
Yard Assembly	65.8	
Transport	3.7	
Installation	1.1	
Offshore Hookup/Commissioning	8.5	203.1
6. Total		379.2

TABLE 5.3.8 ESTIMATED COSTS, 4 JACKETS, 150 FT WATER DEPTH, WEST COAST

	50,000 BOPD	100,000 BOPD
Steel and Fabrication	5.6	8.4
Drilling Facilities	20.0	34.0
Production Facilities	46.0	54.0
Accommodation Module	18.0	24.0
Yard Assembly	26.0	30.8
Transportation	5.2	5.2
Installation	1.1	1.1
Offshore Hookup/Commissioning	8.5	8.5
Total	130.4	166.0

TABLE 5.3.9 A COMPARISON OF FACILITIES COSTS FOR 100,000 AND 50,000 BOPD PRODUCTION RATES, JAPANESE FABRICATION

		$MM
1.	Pile Connectors	
	- Main piles 56 connectors @ 100k	+5.6
	- Skirt Piles 48 @ 117K	+5.6
2.	Varco Type Modules to make up connection	+3.0
3.	Reduced Spread time 40 days welding eliminated +30% = 52 days	
	Derrick Barge @ 110K/day	-5.7
	Material Barge @ 8.6K/day	-0.5
4.	Net Total	+8.0

TABLE 5.3.10 COMPARISON OF COST FOR MECHANICAL CONNECTORS AND WELDING FOR THE PILES

		PRIMARY EARTHQUAKE DIRECTION			DECK LOAD = 60,000 KIPS				DECK LOAD = 16,000 KIPS	
					300 FT WATER		150 FT WATER		150 FT WATER	
		X	Y	Z	DESIGN	API	DESIGN	API	DESIGN	API
1	X	1	2/3	1/2	37.3	22.5	47.5	27.8	14.9	8.7
2	Y	2/3	1	1/2	37.1	22.4	49.7	29.0	13.7	8.0
3	X	1	-	-	31.1	18.7	37.9	22.3	13.1	7.7
4	Y	-	1	-	30.7	18.5	42.7	24.8	10.5	6.2
5	Z	-	-	1	0.1	0.1	5.5	2.4	1.0	0.5

DESIGN: Design Spectrum for this study

API: API Zone 3 Spectrum

X: Along Rows A and B

Y: Along Rows 1, 2, 3, and 4

Z: Vertical

TABLE 5.4.1 <u>SUMMARY OF BASE SHEARS - (RMS) - EARTHQUAKE LOADING x 1000 KIPS</u>

		PRIMARY EARTHQUAKE DIRECTION			DECK LOAD = 60,000 KIPS				DECK LOAD = 16,000 KIPS	
					300 FT WATER		150 FT WATER		150 FT WATER	
		X	Y	Z	DESIGN	API	DESIGN	API	DESIGN	API
1	X	1	2/3	1/2	-4.5	-2.7	-3.9	-2.2	-4.7	-2.8
2	Y	2/3	1	1/2	-4.0	-2.4	-3.6	-2.1	-5.0	-3.0
3	X	1	-	-	-3.8	-2.3	-3.3	-1.9	-3.5	-2.1
4	Y	-	1	-	-2.8	-1.7	-2.6	-1.5	-4.2	-2.5
5	Z	-	-	1	-2.7	-1.4	-2.3	-1.2	-2.5	-1.2
6	Dead Load				-2.6	-2.6	-2.6	-2.6	-1.3	-1.3

TABLE 5.4.2 SUMMARY OF AXIAL PILE FORCES - CORNER PILE (x 1,000 KIPS) - EARTHQUAKE LOAD AXIAL FORCE PER PILE

		PRIMARY EARTHQUAKE DIRECTION			DECK LOAD = 60,000 KIPS			
					300 FT WATER		150 FT WATER	
		X	Y	Z	DESIGN	API	DESIGN	API
1	X	1	2/3	1/2	-3.3	-1.9	-4.6	-1.7
2	Y	2/3	1	1/2	-4.2	-2.5	-4.1	-2.3
3	X	1	-	-	-0.8	-0.5	-0.5	-0.2
4	Y	-	1	-	-3.7	-2.2	-3.6	-2.1
5	Z	-	-	1	-4.0	-2.1	-3.7	-1.9
6	Dead Load				-3.4	-3.4	-3.4	-3.4

TABLE 5.4.3 SUMMARY OF AXIAL PILE FORCES - INTERIOR PILE (x 1,000 KIPS) - EARTHQUAKE LOADING FORCE PER PILE

		300 FT WATER DECK + ICE $= 62,500^K$		150 FT WATER DECK + ICE $= 62,500^K$		DRLG. PLATFORM 150 FT WATER DECK + ICE $= 16,000^K$		PROD. PLATFORM 150 FT WATER DECK + ICE $= 16,000^K$	
		Design	API	Design	API	Design	API	Design	API
1	Basic Steel Structure	19.5	17.5	15.4	13.7	6.5	5.6	5.4	4.5
2	Miscellaneous Items Buoyancy Tank/Mud-Mats, Etc.	1.8	1.8	1.8	1.8	0.8	0.8	0.7	0.7
3	Piles	19.7	15.4	18.2	19.8	7.1	4.8	7.1	4.8
4	Total	41.0	34.7	35.4	38.9	14.4	11.2	13.2	10.0

† Analysis not performed.

- Conductors not included.

TABLE 5.4.4 SUMMARY OF ESTIMATED WEIGHTS - (x 1,000 KIPS)

Translation	1.03×10^6 kips/ft
Vertical	1.28×10^6 kips/ft
Rotation	4.06×10^{10} kip ft/rad.

TABLE 6.1.1 SPRING CONSTANTS FOR SOIL/STRUCTURE INTERACTION

Translation	1.2×10^5 kip sec/ft
Vertical	2.3×10^5 kip sec/ft
Rotation	1.84×10^9 kip ft sec/rad

TABLE 6.1.2 DAMPING CONSTANTS FOR SOIL STRUCTURE INTERACTION

MODE	PERIOD (SEC)
1	1.27
2	1.22
3	0.94
4	0.43
5	0.39
6	0.39
7	0.30
8	0.27
9	0.27
10	0.27

TABLE 6.2.1 LOWEST NATURAL PERIODS FOR 300 FT WATER DEPTH JACKET FIXED TO THE BASE CAISSON

WAVE PERIOD (SEC)	FORCE (KIPS)
12	40,070
14	71,850
16	93,520
18	105,450

Notes: Water Depth 300 ft.
Wave Height 71 ft.

TABLE 6.2.2 MAXIMUM HORIZONTAL WAVE LOAD ON HYBRID STRUCTURE BASE CAISSON AT THE MUDLINE AS A FUNCTION OF WAVE PERIOD

WAVE PERIOD (SEC)	FORCE (KIPS)
12	65,260
14	114,960
16	157,819
18	189,187

Notes: Water Depth 300 ft.
Wave Height 71 ft.

TABLE 6.2.3 MAXIMUM VERTICAL WAVE LOADS ON HYBRID STRUCTURE BASE CAISSON AT THE MUDLINE AS A FUNCTION OF WAVE PERIOD

WAVE PERIOD (SEC)	MOMENT (KIP.FT.)
12	4.30×10^6
14	4.16×10^6
16	3.69×10^6
18	3.23×10^6

Notes: Water Depth 300 ft.
Wave Height 71 ft.

TABLE 6.2.4 MAXIMUM OVERTURNING MOMENTS ON HYBRID STRUCTURE BASE CAISSON AT THE MUDLINE AS A FUNCTION OF WAVE PERIOD

MODE	PERIOD (SEC)
1	0.67
2	0.65
3	0.53
4	0.26
5	0.24
6	0.24
7	0.22
8	0.21
9	0.21
10	0.19

TABLE 6.2.5 LOWEST NATURAL PERIODS FOR 150 FT WATER DEPTH JACKET FIXED TO THE BASE CAISSON

WAVE PERIOD (SEC)	FORCE (KIPS)
12	25,960
14	101,650
16	140,000
18	158,250

Notes: Water Depth 150 ft.
Wave Height 71 ft.

TABLE 6.2.6 MAXIMUM HORIZONTAL FORCE ON HYBRID PLATFORM BASE CAISSON AT THE MUDLINE FROM WAVE LOADS IN 150 FT WATER DEPTH

WAVE PERIOD (SEC)	FORCE (KIPS)
12	61,540
14	151,600
16	270,140
18	378,000

Notes: Water Depth 150 ft.
Wave Height 71 ft.

TABLE 6.2.7 MAXIMUM VERTICAL FORCE ON HYBRID SYSTEM BASE CAISSON AT THE MUDLINE FROM WAVE LOADS IN 150 FT WATER DEPTH

PERIOD (SEC)	FORCE (FT.KIPS)
12	3.09×10^7
14	3.28×10^7
16	2.97×10^7
18	2.68×10^7

Notes: Water Depth 150 ft.
Wave Height 71 ft.

TABLE 6.2.8 MAXIMUM OVERTURNING MOMENT ON HYBRID PLATFORM BASE CAISSON AT THE MUDLINE FROM WAVE LOADS IN 150 FT WATER DEPTH

Significant Wave Height	20 ft
Peak Period	12 secs.
JONSWAP Spectrum	

TABLE 6.2.9 DESIGN TOW SEASTATE

Direction	Single Amplitude* Displacement
Heave	8.47 ft
Surge	23.85 ft
Pitch	2.86 deg

* Displacement measured at the corner of the deck.

TABLE 6.2.10 SINGLE AMPLITUDE 1/1000 DISPLACEMENT UNDER THE DESIGN TOW SEASTATE

Direction	Velocity*
Heave	4.39 ft/sec
Surge	12.75 ft/sec
Pitch	1.55 deg/sec

* Velocity measured at the corner of the deck.

TABLE 6.2.11 1/1000 VELOCITIES UNDER THE DESIGN TOW SEASTATE

Direction	Accellerations*
Heave	2.33 ft/sec^2
Surge	6.89 ft/sec^2
Pitch	0.85 $deg./sec^2$

* Accelerations measured at the corner of the deck.

TABLE 6.2.12 1/1000 ACCELERATIONS UNDER DESIGN TOW SEASTATE

ITEM	UNIT	COST ($U.S.) U.S.	JAPAN
Concrete: Normal	cu.yd.	155	147
Lightweight		222	212
Reinforcement	s.ton	1,325	932
Post Tensioning	s.ton	4,540	3,820
Formwork	sq.ft.	5.34	4.66
Concrete Placing Rate	cu.yd./wk	2,000	3,450
Rebar Placing Rate	s.ton/wk	400	440
Post Tensioning Rate	s.ton/wk	50	55
Formwork Rate	sq.ft/wk	30,000	32,300

TABLE 6.3.1 UNIT RATES FOR REINFORCED CONCRETE IN JAPAN AND WEST COAST

		($MM)	
1.	Engineering & Management		
	Marine Insurance (10%)		33.9
2.	Steel Jacket		
	Materials	5.2	
	Fabrication	9.5	
	Tow to Mating Site	0.3	15.0
3.	Deck		
	Materials	1.5	
	Fabrication	4.5	
	Tow to Mating Site	0.3	
4.	Concrete Structure		
	Construction Site (4 Uses)	3.3	
	Skirts	1.0	
	Base Slab	12.3	
	Walls	34.3	
	Tow to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	6.4	
	Hull Systems	25.0	
	Tow to Mating Site	0.3	86.6
5.	Modules		
	Steel and Fabrication	8.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	24.0	
	Yard Assembly	30.8	
	Tow to Mating Site	1.3	152.5

TABLE 6.3.2 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 150 FT WATER DEPTH, 100,000 BOPD (SHEET 1 OF 2)

		($MM)
6. Mating: Inshore Location		
Concrete Base to Jack.	1.7	
Install Modules	0.8	
Hookup	5.1	7.6
7. Tow to Alaska		
Prepare for Tow	1.0	
Barite Ballast:		
Eqpt.	4.1	
Mat'l.	17.2	
Tow	11.0	33.3
8. Install & Commission		
Water Ballast	3.2	
Grout Under Base:		
Mat'l	6.6	
Eqpt.	4.0	
Barite Ballast:		
Mat'l	12.6	
Eqpt.	3.3	
Diving Support	6.7	
Commission	1.0	
Demob Equipment	1.3	38.7
9. Total		373.3

TABLE 6.3.2 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 150 FT WATER DEPTH, 100,000 BOPD (SHEET 2 OF 2)

		($MM)	
1.	Engineering & Management Marine Insurance (10%)		37.7
2.	Steel Jacket		
	Materials	7.7	
	Fabrication	13.7	
	Tow to Mating Site	0.3	21.7
3.	Deck		
	Materials	1.5	
	Fabrication	4.5	
	Tow to Mating Site	0.3	6.3
4.	Concrete Structure		
	Construction Site (4 Uses)	3.3	
	Skirts	0.6	
	Base Slab	12.6	
	Walls	63.8	
	Tow to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	6.5	
	Hull Systems	27.0	
	Tow to Mating Site	0.3	118.1
5.	Modules		
	Steel and Fabrication	8.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	24.0	
	Yard Assembly	30.8	
	Tow to Mating Site	1.3	152.5

TABLE 6.3.3 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 300 FT WATER DEPTH, 100,000 BOPD (SHEET 1 OF 2)

			($MM)
6.	Mating: Inshore Location		
	Concrete Base to Jack.	1.7	
	Install Modules	0.8	
	Hookup	5.1	7.6
7.	Tow to Alaska		
	Prepare for Tow	1.0	
	Barite Ballast:		
	Eqpt.	2.8	
	Mat'l.	11.6	
	Tow	11.0	26.4
8.	Install & Commission		
	Water Ballast	3.2	
	Grout Under Base:		
	Mat'l	6.0	
	Eqpt.	3.8	
	Barite Ballast:		
	Mat'l	15.4	
	Eqpt.	7.0	
	Diving Support	6.7	
	Commission	1.0	
	Demob Equipment	1.3	44.4
9.	Total		414.7

TABLE 6.3.3 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 300 FT WATER DEPTH, 100,000 BOPD (SHEET 2 OF 2)

		($MM)	
1.	Engineering & Management		
	Marine Insurance (10%)		39.4
2.	Steel Jacket		
	Materials	5.2	
	Fabrication	11.6	
	Tow to Mating Site	0.3	17.1
3.	Deck		
	Materials	1.5	
	Fabrication	5.3	
	Tow to Mating Site	0.3	7.1
4.	Concrete Structure		
	Construction Site (4 Uses)	3.3	
	Skirts	1.3	
	Base Slab	15.3	
	Walls	42.5	
	Tow to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	7.9	
	Hull Systems	25.0	
	Tow to Mating Site	0.3	99.6
5.	Modules		
	Steel and Fabrication	9.0	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	27.0	
	Yard Assembly	65.8	
	Tow to Mating Site	1.3	191.1

TABLE 6.3.4 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN WEST COAST, 150 FT WATER DEPTH, 100,000 BOPD (SHEET 1 OF 2)

		($MM)
6. Mating: Inshore Location		
Concrete Base to Jack.	1.1	
Install Modules	0.8	
Hookup	6.9	8.8
7. Tow to Alaska		
Prepare for Tow	1.0	
Barite Ballast:		
Eqpt.	4.1	
Mat'l.	17.2	
Tow	9.0	31.3
8. Install & Commission		
Water Ballast	3.2	
Grout Under Base:		
Mat'l	6.6	
Eqpt.	4.0	
Barite Ballast:		
Mat'l	12.6	
Eqpt.	3.3	
Diving Support	6.7	
Commission	1.0	
Demob Equipment	1.3	38.7
9. Total		433.1

TABLE 6.3.4 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN WEST COAST, 150 FT WATER DEPTH, 100,000 BOPD (SHEET 2 OF 2)

			($MM)
1.	Engineering & Management		
	Marine Insurance (10%)		43.9
2.	Steel Jacket		
	Materials	7.7	
	Fabrication	16.7	
	Tow to Mating Site	0.3	24.7
3.	Deck		
	Materials	1.5	
	Fabrication	5.3	
	Tow to Mating Site	0.3	7.1
4.	Concrete Structure		
	Construction Site (4 Uses)	3.3	
	Skirts	0.8	
	Base Slab	15.7	
	Walls	79.1	
	Tow to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	8.0	
	Hull Systems	27.0	
	Tow to Mating Site	0.3	138.2
5.	Modules		
	Steel and Fabrication	9.0	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	27.0	
	Yard Assembly	65.8	
	Tow to Mating Site	1.3	191.1

TABLE 6.3.5 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN WEST COAST, 300 FT WATER DEPTH, 100,000 BOPD (SHEET 1 OF 2)

		($MM)
6. Mating: Inshore Location		
Concrete Base to Jack.	1.7	
Install Modules	0.8	
Hookup	6.9	9.4
7. Tow to Alaska		
Prepare for Tow	1.0	
Barite Ballast:		
Eqpt.	2.8	
Mat'l.	11.6	
Tow	9.0	24.4
8. Install & Commission		
Water Ballast	3.2	
Grout Under Base:		
Mat'l	6.0	
Eqpt.	3.8	
Barite Ballast:		
Mat'l	15.4	
Eqpt.	7.0	
Diving Support	6.7	
Commission	1.0	
Demob Equipment	1.3	44.4
9. Total		483.2

TABLE 6.3.5 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN WEST COAST, 300 FT WATER DEPTH, 100,000 BOPD (SHEET 2 OF 2)

		($MM)
1. Engineering & Management Marine Insurance (10%)		26.2
2. Steel Jacket		
Materials	5.2	
Fabrication	9.5	
Tow to Mating Site	0.3	15.0
3. Deck		
Materials	1.5	
Fabrication	4.5	
Tow to Mating Site	0.3	6.3
4. Concrete Structure		
Construction Site (4 Uses)	3.3	
Skirts	0.5	
Base Slab	6.2	
Walls	19.7	
Tow to Deeper Water	1.0	
Temporary Moorings	3.0	
Top Slab	3.5	
Hull Systems	25.0	
Tow to Mating Site	0.3	62.5
5. Modules		
Steel and Fabrication	5.6	
Drilling Facilities	20.0	
Production Facilities	46.0	
Quarters Facilities	18.0	
Yard Assembly	26.0	
Tow to Mating Site	1.3	116.9

TABLE 6.3.6 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 150 FT WATER DEPTH, 50,000 BOPD (SHEET 1 OF 2)

			($MM)
6.	Mating: Inshore Location		
	Concrete Base to Jack.	1.7	
	Install Modules	0.8	
	Hookup	4.6	7.1
7.	Tow to Alaska		
	Prepare for Tow	1.0	
	Barite Ballast:		
	Eqpt.	2.3	
	Mat'l.	9.7	
	Tow	11.0	23.0
8.	Install & Commission		
	Water Ballast	3.2	
	Grout Under Base:		
	Mat'l	3.8	
	Eqpt.	2.8	
	Barite Ballast:		
	Mat'l	8.9	
	Eqpt.	2.8	
	Diving Support	6.7	
	Commission	1.0	
	Demob Equipment	1.3	30.5
9.	Total		287.5

TABLE 6.3.6 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 150 FT WATER DEPTH, 50,000 BOPD (SHEET 2 OF 2)

		($MM)	
1.	Engineering & Management		
	Marine Insurance (10%)		29.5
2.	Steel Jacket		
	Materials	7.7	
	Fabrication	13.7	
	Tow to Mating Site	0.3	21.7
3.	Deck		
	Materials	1.5	
	Fabrication	4.5	
	Tow to Mating Site	0.3	6.3
4.	Concrete Structure		
	Construction Site (4 Uses)	3.3	
	Skirts	0.5	
	Base Slab	9.1	
	Walls	42.3	
	Tow to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	4.7	
	Hull Systems	27.0	
	Tow to Mating Site	0.3	91.2
5.	Modules		
	Steel and Fabrication	5.6	
	Drilling Facilities	20.0	
	Production Facilities	46.0	
	Quarters Facilities	18.0	
	Yard Assembly	26.0	
	Tow to Mating Site	1.3	116.9

TABLE 6.3.7 <u>COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 300 FT WATER DEPTH, 50,000 BOPD (SHEET 1 OF 2)</u>

		($MM)
6. Mating: Inshore Location		
Concrete Base to Jack.	1.7	
Install Modules	0.8	
Hookup	5.1	7.6
7. Tow to Alaska		
Prepare for Tow	1.0	
Barite Ballast:		
Eqpt.	1.6	
Mat'l.	6.6	
Tow	11.0	20.0
8. Install & Commission		
Water Ballast	3.2	
Grout Under Base:		
Mat'l	3.4	
Eqpt.	2.2	
Barite Ballast:		
Mat'l	10.9	
Eqpt.	2.7	
Diving Support	6.7	
Commission	1.0	
Demob Equipment	1.3	31.4
9. Total		324.8

TABLE 6.3.7 COST ESTIMATE FOR HYBRID STRUCTURE, FABRICATED IN JAPAN, 300 FT WATER DEPTH, 50,000 BOPD (SHEET 2 OF 2)

	150'	300'
Caisson Size	425' x 425' x 90'	366' x 366' x 120'
Concrete Volume	73,000 yd^3	111,000 yd^3
Barite		
Tow	572,000 kips (15 ft)	385,000 kips (15 ft)
Site	420,000 kips (17 ft)	514,000 kips (30 ft)
Effective Weight	877,000 $\pm$ 210,000 kips	880,000 $\pm$ 180,000 kips
Minimum GM	18 ft	11.9 ft
Jacket Main Framing	9,500 kips	14,600 kips

TABLE 6.4.1 COMPARISON OF 150 FT AND 300 FT WATER DEPTH HYBRID STRUCTURES

	50,000 BOPD	100,000 BOPD
Caisson Size	276' x 276' x 120'	366' x 366' x 120'
Concrete Volume	63,000 yd^3	111,000 yd^3
Barite		
Tow	218,000 kips (15 ft)	385,000 kips (15 ft)
Site	363,000 kips	514,000 kips
Effective Weight	556,000 $\pm$ 129,000 kips	880,000 $\pm$ 180,000 kips
Minimum GM	2.0 ft	11.9 ft
Jacket Main Framing	16,000 kips	14,600 kips

TABLE 6.4.2 COMPARISON OF 100,000 AND 50,000 BOPD PRODUCTION RATE HYBRID STRUCTURES

	API-RP2A	DESIGN
Caisson Size	325' x 325' x 120'	366' x 366' x 120'
Concrete Volume	87,000 yd^3	111,000 yd^3
Barite		
Tow	199,000 kips (10 ft)	385,000 kips (15 ft)
Site	229,000 kips	514,000 kips (10 ft)
Effective Weight	512,000 $\pm$ 105,000 kips	880,000 $\pm$ 180,000 kips
Minimum GM	6.4 ft	11.9 ft
Jacket Main Framing	11,000 kips	14,600 kips

TABLE 6.4.3 COMPARISON OF API, ZONE 3 AND DESIGN RESPONSE SPECTRUM HYBRID STRUCTURES

MATERIAL	SPECIFIED STRENGTH (KSI)	UNIT WEIGHT (LB/FT^3)
Concrete	7	140
Reinforcement	60	490
Prestessing	270	490
Placed Reinforced Concrete (assumed)	-	170

TABLE 7.0.1 MATERIAL UNIT PROPERTIES FOR CONCRETE GRAVITY SYSTEM

Water Depth	300 ft
Production Rate	100,000 bpd
Deck Weight	60,000 kips
Design Spectrum	Figure 2.3.4

TABLE 7.1.1 BASE CASE CONDITIONS FOR THE CONCRETE GRAVITY PLATFORM

LOADING CONDITION	MAXIMUM SHEAR (KIPS)	MAXIMUM MOMENT (KIP.FT)
Earthquake	22,046	2.77×10^6
Wave	12,763	1.20×10^6

TABLE 7.2.1 A COMPARISON OF WAVE AND SEISMIC LOADS AT THE TOWER BASE

Axial Load	73,000 kips
Moment	2.77×10^6 kip ft
Concrete	7,000 psi (28 day)
Reinforcement	1.0%
Prestressing	510 psi

TABLE 7.2.2 DESIGN LOADS AND REQUIREMENTS FOR THE PRODUCTION TOWERS

COMPONENT	CRITICAL CONDITION	UNFACTORED LOAD	LOAD FACTOR
Outer Wall	Implosion at Deck-Setting	22.7 ksf	1.2
Interior Walls	Hydrostatic Head from Damaged Condition at Deck Setting	22.7 ksf	1.1
Base Slab	Hydrostatic Head + Hardspot at Installation	19.2 + 10.0 ksf	1.2
Top Slab	Hydrostatic Head at Deck-Setting	15.0 ksf	1.2

TABLE 7.2.3 SUMMARY OF DESIGN CONDITIONS FOR LOCAL DESIGN OF THE CAISSON

Production Rate	100,000 bpd
Water Depth	300 ft.
Deck Weight	60,000 kips
Crude Storage	900,000 barrels
No. of Rigs	2
Base Caisson	366 x 366 x 120 ft
Tower Height	255 ft
Concrete Volume	132,122 yd^3
Barite/Towing	129,422 tons (10 ft)
Additional Barite (site)	284,729 tons (22 ft)
Effective Weight	861,000 kips
Maximum Base Shear	340,000 kips
Minimum G.M.	2.0 ft

TABLE 7.2.4 BASE CASE PARAMETERS FOR THE CONCRETE GRAVITY STRUCTURE

W.D. (ft)	W_{eff} ($x10^3$ kip)	CASE	H ($x10^3$ kip)	V ($x10^3$ kip)	M ($x10^6$ kip ft)
300	880	Seismic: Max H & Down	350	180	-38
		Seismic: Max Up	320	-170	-36
		Seismic: Max Moment M	220	-100	-46
150	864	Seismic: Max H	330	-200	-23
		Seismic: Max Down	300	210	-25
		Seismic: Max Up	300	-210	-12
		Seismic: Max Moment M	290	200	-38
		Wave: Max H & M (wt=70°)	180	-129.97	-25
		Wave: Max Down (wt=40°)	-61.56	380	+8.55
		Wave: Max Up (wt=140°)	61.56	-380	-8.55
		Wave: Max H & M (wt=110°)	-180	129.97	+25

Note:
- H: Horizontal load
- V: Vertical load
- M: Moment
- Wt: Phase angle

TABLE 7.3.1 ENVELOPE LOADING CONDITIONS FOR GEOTECHNICAL STABILITY OF HYBRID AND CONCRETE GRAVITY SYSTEMS

W.D. (ft)	W_{EFF} ($x10^3$ kip)	V ($x10^3$kip)	d_s (ksf)	RESIST. ($x10^3$kip)	H ($x10^3$kip)	F.S.
300	800	180	0.42	784.69	350	2.24
		-170		539.61	320	1.69
		-100		588.63	220	2.68
150	864	-200	0.42	507.40	330	1.54
		210		794.49	300	2.65
		-210		500.40	300	1.67
		-200		507.40	290	1.75
		-129.97		556.44	180	3.09
		380		913.52	-61.56	14.84
		-380		381.37	61.56	6.20
		129.97		738.45	-180	4.10

Notes:

W.D.:	Water depth
W_{eff}:	Effective weight
V:	Vertical load
H:	Horizontal load
ds:	Overburden to skirt tip
Resist:	Sliding resistence at the skirt tip
F.S.:	Factor of safety

TABLE 7.3.2 SHALLOW SLIDING STABILITY RESULT SUMMARY

W.D. (ft)	CASE	W_{EFF} ($x10^3$kip)	H ($x10^3$kip)	V ($x10^3$kip)	M ($x10^6$ft.kip)	F.S.
300	S: Max H & Down	880	350	180	-38	1.29
	S: Max Up	880	320	-170	-36	1.40
	S: Max M	880	220	-100	-46	2.03
150	S: Max H	864	330	-200	-23	1.33
	S: Max Down	864	300	210	-25	1.45
	S: Max Up	864	300	-210	-12	1.42
	S: Max M	864	290	-200	-38	1.54
	W: Max H & Mom (wt=70°)	864	180	-129.97	-25	2.30
	W: Max Down (wt=40°)	864	61.56	380	-8.55	4.18
	W: Max Up (wt=140°)	864	61.56	-380	-8.55	5.22
	W: Max H & Mom (wt=110°)	864	180	129.97	-25	2.25

Notes:

Wt:	Phase angle
H:	Horizontal load
V:	Vertical load
M:	Moment
W_{eff}:	Effective weight
W.D.:	Water depth
S:	Seismic load
W:	Wave load
F.S.:	Factor of safety

TABLE 7.3.3 DEEP SLIDING MODE FACTORS OF SAFETY, PROFILE A

W.D. (ft)	CASE	W_{EFF} ($x10^3$kip)	H ($x10^3$kip)	V ($x10^3$kip)	M ($x10^6$ft.kip)	F.S.
300	S: Max H & Down	880	350	180	-38	1.30
	S: Max Up	880	320	-170	-36	1.49
	S: Max M	880	220	-100	-46	1.94
150	S: Max H	864	330	-200	-23	1.39
	S: Max Down	864	300	210	-25	1.34
	S: Max Up	864	300	-210	-12	1.42
	S: Max M	864	290	-200	-38	1.63
	W: Max H & Mom (wt=70°)	864	180	-129.97	-25	1.93
	W: Max Down (wt=40°)	864	61.56	380	-8.55	1.94
	W: Max Up (wt=140°)	864	61.56	-380	-8.55	2.86
	W: Max H & Mom (wt=110°)	864	180	129.97	-25	1.75

TABLE 7.3.4 DEEP SLIDING MODE FACTORS OF SAFETY, PROFILE B

			($MM)
1.	Engineering & Management		
	Marine Insurance (10%)		32.7
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	1.0	
	Base Slab	12.3	
	Walls	34.3	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	6.4	
	Towers	5.4	
	Hull Systems	25.0	
	Transport to Mating Site	0.7	89.1
4.	Topsides		
	Deck Fabrication	14.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	24.0	
	Yard Assembly	30.8	
	Transport and Mate	6.3	163.5
5.	Tow to Field (Incl. part barrite)		28.3
6.	Install & Commission		42.6
7.	Total		359.5

TABLE 7.4.1 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, JAPAN, 100,000 BOPD, 150 FT WATER DEPTH

			($MM)
1.	Engineering & Management		
	Marine Insurance (10%)		35.7
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	0.6	
	Base Slab	12.6	
	Walls	63.8	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	6.5	
	Towers	9.6	
	Hull Systems	27.0	
	Transport to Mating Site	0.7	124.8
4.	Topsides		
	Deck Fabrication	14.4	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	24.0	
	Yard Assembly	30.8	
	Transport and Mate	6.3	163.5
5.	Tow to Field (Incl. part barrite)		21.7
6.	Install & Commission		43.5
7.	Total		392.5

TABLE 7.4.2 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, JAPAN, 100,000 BOPD, 300 FT WATER DEPTH

			($MM)
1.	Engineering & Management Marine Insurance (10%)		37.9
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	1.2	
	Base Slab	15.3	
	Walls	42.5	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	7.9	
	Towers	6.7	
	Hull Systems	25.0	
	Transport to Mating Site	0.7	103.3
4.	Topsides		
	Deck Fabrication	16.2	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	27.0	
	Yard Assembly	65.8	
	Transport and Mate	6.3	203.3
5.	Tow to Field (Incl. part barrite)		26.3
6.	Install & Commission		42.6
7.	Total		416.7

TABLE 7.4.3 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, WEST COAST, 100,000 BOPD, 150 FT WATER DEPTH

			($MM)
1.	Engineering & Management Marine Insurance (10%)		41.7
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	0.8	
	Base Slab	15.7	
	Walls	79.1	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	8.0	
	Towers	11.9	
	Hull Systems	27.0	
	Transport to Mating Site	0.7	147.2
4.	Topsides		
	Deck Fabrication	16.2	
	Drilling Facilities	34.0	
	Production Facilities	54.0	
	Quarters Facilities	27.0	
	Yard Assembly	65.8	
	Transport and Mate	6.3	203.3
5.	Tow to Field (Incl. part barrite)		19.7
6.	Install & Commission		43.5
7.	Total		458.7

TABLE 7.4.4 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, WEST COAST, 100,000 BOPD, 300 FT WATER DEPTH

			($MM)
1.	Engineering & Management Marine Insurance (10%)		26.7
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	0.5	
	Base Slab	6.2	
	Walls	19.7	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	3.5	
	Towers	5.4	
	Hull Systems	25.0	
	Transport to Mating Site	0.7	65.0
4.	Topsides		
	Deck Fabrication	12.0	
	Drilling Facilities	20.0	
	Production Facilities	54.0	
	Quarters Facilities	18.0	
	Yard Assembly	26.0	
	Transport and Mate	6.3	136.3
5.	Tow to Field (Incl. part barrite)		26.5
6.	Install & Commission		35.6
7.	Total		293.4

TABLE 7.4.5 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, JAPAN, 50,000 BOPD, 150 FT WATER DEPTH

			($MM)
1.	Engineering & Management Marine Insurance (10%)		29.0
2.	Construction Site (4 Uses)		3.3
3.	Concrete Structure		
	Skirts	0.5	
	Base Slab	9.1	
	Walls	42.3	
	Transport to Deeper Water	1.0	
	Temporary Moorings	3.0	
	Top Slab	4.7	
	Towers	9.6	
	Hull Systems	27.0	
	Transport to Mating Site	0.7	97.9
4.	Topsides		
	Deck Fabrication	12.0	
	Drilling Facilities	20.0	
	Production Facilities	54.0	
	Quarters Facilities	18.0	
	Yard Assembly	26.0	
	Transport and Mate	6.3	136.3
5.	Tow to Field (Incl. part barrite)		21.7
6.	Install & Commission		31.0
7.	Total		319.2

TABLE 7.4.6 COST ESTIMATE FOR CONCRETE GRAVITY PLATFORM, JAPAN, 50,000 BOPD, 300 FT WATER DEPTH

Case 1:	Water Depth	150 ft
	Production Rate	100,000 bpd
Case 2:	Water Depth	300 ft
	Production Rate	50,000 bpd

TABLE 7.5.1 SENSITIVITY CASES CONSIDERED FOR CONCRETE GRAVITY PLATFORM

PRODUCTION RATE	50,000 BOPD		100,000 BOPD	
Water Depth (Ft)	150	300	150	300
Deck Weight (kips)	50,000		60,000	
Crude Storage (Bbls)	450,000		900,000	
No. of Conductors	36		56	
No. of Rigs	1		2	
Effective Caisson Size L x B x H	350'x350'x80'	300'x300'x120'	454'x454'x80'	366'x366'x120'
Tower Height (Ft)	145	255	145	255
Concrete Volume (Yd^3)	52,250	92,600	81,467	132,122
Displaced Volume (Ft^3)	1.05×10^7	1.25×10^7	1.72×10^7	1.76×10^7
Req'd Barite (Tons)	194,922	130,572	218,702	129,422
Additional Barite at Site (Tons)	194,922	174,096	262,443	284,729
Effective Weight (kips)	662,000	583,000	858,000	861,000
Minimum GM (Ft)	6.8	2.0	10.4	2.0

TABLE 7.5.2 COMPARISON OF 150 FT AND 300 FT W.D. CONCRETE GRAVITY PLATFORMS AT TWO PRODUCTION RATES

MODE	BASE CASE (SEC)	LIQUEFACTION CASE (SEC)
1	1.82	2.34
2	1.81	2.26
3	1.34	1.82
4	0.57	0.89
5	0.56	0.63

TABLE 8.3.1 A COMPARISON OF THE LOWEST NATURAL PERIODS WITH AND WITHOUT LIQUEFACTION FOR THE JACKET IN 300 FT WATER DEPTH

MODE	BASE CASE (SEC)	LIQUEFACTION CASE (SEC)
1	1.30	1.62
2	1.07	1.57
3	0.87	1.31
4	0.41	0.71
5	0.35	0.57

TABLE 8.3.2 A COMPARISON OF THE LOWEST NATURAL PERIODS WITH AND WITHOUT LIQUEFACTION IN THE 150 FT WATER DEPTH, MULIPLE JACKET CASE

	BASE CASE (KIPS)	WITH LIQUEFACTION (KIPS)
300 ft Depth	37,300	32,200
150 ft Depth (Multiple Jackets)	14,900	12,300

TABLE 8.3.3 A COMPARISON OF MAXIMUM BASE SHEARS ON JACKETS (RMS IN KIPS) FOR CASES WITH AND WITHOUT LIQUEFACTION

	BASE CASE (KIPS)	WITH LIQUEFACTION (KIPS)
Exterior Pile	4,500	4,100
Interior Pile	3,300	3,100

TABLE 8.3.4 A COMPARISON OF PILE AXIAL LOADS IN KIPS FOR THE JACKET IN 300 FT OF WATER

DESCRIPTION	BASE CASE	WITH LIQUEFACTION
Basic Steel Structure	19,500	21,200
Miscellaneous Items	1,800	1,800
Piles	19,700	21,200
Total	41,000	44,200

TABLE 8.3.5 SUMMARY OF ESTIMATED WEIGHTS (KIPS) FOR JACKET IN 300 FT OF WATER

DESCRIPTION	BASE CASE	WITH LIQUEFACTION
Basic Steel Structure	15,400	17,300
Miscellaneous Items	1,800	1,800
Piles	18,200	19,800
Total	35,400	38,900

TABLE 8.3.6 SUMMARY OF ESTIMATED WEIGHTS IN KIPS FOR THE 150 FT WATER DEPTH CASE (SINGLE JACKET)

DESCRIPTION	BASE CASE	WITH LIQUEFACTION
Basic Steel Structure	6,500	7,500
Miscellaneous Items	800	900
Piles	7,100	7,700
Total	14,400	15,600

TABLE 8.3.7 SUMMARY OF ESTIMATED WEIGHTS IN KIPS FOR THE 150 FT WATER DEPTH CASE (MULTIPLE JACKETS)

ITEM	COST ($MM)	
1. Engineering/Management Marine Insurance (10%)		16.3
2. Production Tanker		
Purchase	7.5	
Modifications	21.9	
Outfitting	59.0	
Ice Strengthening	15.0	
Shipyard Fee	7.2	
Tow to Location	0.7	
Installation	0.2	111.5
3. Mooring System		
Materials & Fabrication	4.2	
Transportation	0.7	
Installation	1.6	6.5
4. Subsea System		
Materials & Fabrication	22.8	
Transportation	1.2	
Installation	4.8	28.8
5. Risers		
Materials & Fabrication	9.6	
Transportation	0.7	
Installation	1.7	12.0
6. Test & Commission		3.5
TOTAL		178.6

TABLE 9.3.1 FPSO COST ESTIMATE

Deck Strengthening

Keel Strengthening

Turret Section

Painting and Cathodic Protection

Equipment foundations

Yoke Fortification

Accommodations Upgrade

Cover for Process Equipment

Shuttle Tanker Mooring Capability

TABLE 9.3.2 <u>ITEMS INCLUDED IN HULL MODIFICATIONS, TABLE 9.3.1</u>

Process Equipment (See Table 9.3.4)

Metering Facility

Pumping

Piping

Stationkeeping

Generators

Emergency generators

Fire Fighting

Safety Shutdown

Communications/Visual Aids

Deck Cranes

Lifeboats

Modifications and Repairs

Equipment Installation/Removal (See Table 9.3.5)

Turret

Swivel

TABLE 9.3.3 <u>HULL OUTFIT ITEMS IN TABLE 9.3.1</u>

DESCRIPTION	QTY
1st Stage Separator	2
2nd Stage Separator	2
Test Separator	1
Heat Exchanger	2
Prod. Wtr. Performax	1
Prod. Wtr. Dual Filter	2
Flash Stabilizer	2
Surge Tank	1
SW Treatment Tank	2
SW Dual Filter	2
Fuel Gas Skid	1
Oil Metering Facil.	1
Flare Vent. Separ.	1
Water Inject. Pumps	2
Gas Compress Cool.	2
Compress Consoles	1
SW Lift Pumps	2
Inlet Manifold	1
Gas Inj. Manifold	1
Wtr Inj. Manifold	1
Flare Boom	2
Flare Gas K.O. Drum	1
Mooring/Loading Facil.	1
Air Compress. Pack	1
Emerg. Generator	1

TABLE 9.3.4 SPECIFIC EQUIPMENT INCLUDED IN THE PROCESS EQUIPMENT COST ESTIMATE

DESCRIPTION	QTY
SCR Room	1
Instrumentation	N/A
Elect. Workshop	1
Mech. Workshop	1
Battery Room	1
Transformer Room	1
Emerg. Control Room	1
Main Control Room	1
FWD Escape Caps	2
Electrical	N/A
Accommodations	1
H & V Plant Room	1
Aft Crane	1
Elect. Switch Room	1
Turret Facility	1
Mooring Equipment	N/A
Aft Crane	1
Piping	N/A
Main Power Gen.	3
Riser Handling	1

ADDED

DESCRIPTION	QTY
Tail Shaft	1
Main Screw	1
Rudder	1
Rudder Shaft	1
Anchor	2
Anchor Chain	2
Main Engine	1

REMOVED

TABLE 9.3.5 ADDITIONAL EQUIPMENT ADDED AND REMOVED FROM THE FPSO

ITEM	COST ($MM)	
1. Engineering/Management		
Marine Insurance (10%)		9.0
2. Storage Tanker		
Purchase	7.5	
Modifications	13.3	
Outfitting	39.6	
Ice Strengthening	15.0	
Shipyard Fee	4.3	
Tow to Location	0.7	
Installation	0.2	80.5
3. Mooring System		
Materials & Fabrication	4.2	
Transportation	0.7	
Installation	1.6	6.5
4. Risers		
Materials & Fabrication	0.9	
Transportation	0.8	
Installation	0.8	2.5
5. Test & Commission		0.3
TOTAL		98.8

TABLE 9.3.6 FSO COST ESTIMATE

ITEM	100 MBOPD ($MM)		50 MBOPD ($MM)	
1. Engineering/Management Marine Insurance (10%)		16.3		14.9
2. Production Tanker				
Purchase	7.5		7.5	
Modifications	21.9		21.9	
Outfitting	59.0		49.0	
Ice Strengthening	15.0		15.0	
Shipyard Fee	7.2		6.2	
Tow to Location	0.7		0.7	
Installation	0.2	111.5	0.2	100.5
3. Mooring System				
Materials & Fabrication	4.2		4.2	
Transportation	0.7		0.7	
Installation	1.6	6.5	1.6	6.5
4. Subsea System				
Materials & Fabrication	22.8		22.8	
Transportation	1.2		1.2	
Installation & Pile	4.8	28.8	4.8	28.8
5. Risers				
Materials & Fabrication	9.6		6.8	
Transportation	0.7		0.7	
Installation	1.7	12.0	1.7	9.2
6. Test & Commission		3.5		3.5
TOTAL		178.6		163.4

TABLE 9.4.1 FPSO COST ESTIMATE SENSITIVITY STUDY

ITEM	100 MBOPD ($MM)		50 MBOPD ($MM)	
1. Engineering/Management Marine Insurance (10%)		9.0		8.9
2. Production Tanker				
Purchase	7.5		7.5	
Modifications	13.3		13.3	
Outfitting	39.6		39.6	
Ice Strengthening	15.0		15.0	
Shipyard Fee	4.3		4.3	
Tow to Location	0.7		0.7	
Installation	0.2	80.5	0.2	80.5
3. Mooring System				
Materials & Fabrication	4.2		4.2	
Transportation	0.7		0.7	
Installation	1.6	6.5	1.6	6.5
4. Riser/Riser Base				
Materials & Fabrication	0.9		0.4	
Transportation	0.8		0.8	
Installation	0.8	2.5	0.6	1.8
5. Test & Commission		0.3		0.3
TOTAL		98.8		98.0

TABLE 9.4.2 FSO COST ESTIMATE SENSITIVITY STUDY

ITEM	COST ($MM)	
1. Engineering/Management		
Marine Insurance (10%)		0.7
2. Mooring System		
Materials & Fabrication	0.4	
Transportation	0.6	
Installation	1.6	2.6
3. Risers		
Materials & Fabrication	2.5	
Transportation	0.6	
Installation	0.6	3.7
4. Test & Commission		0.2
TOTAL		7.2

TABLE 10.3.1 REMOTE LOADING BUOY COST ESTIMATE

TASK	TIME REQUIREMENT	LIMITING WAVE STATE (SIGNIFICANT)	LIMITATIONS
Mooring	3 Hrs	8 ft	1. During Daylight 2. 2 miles visibility 3. Sufficient Crude Available in Storage
Pre-Loading	3 Hrs	14 ft	1. Always Occurs After Mooring Task
Loading	15 Hrs	14 ft	1. Does not need to be continuous 15 hrs.
Departure	3 Hrs	26 ft	1. Only after tanker is fully loaded.
Stand-by	None	None	-
Transiting	228 Hrs	None	1. Time before it is available again.
Waiting	None	None	-

TABLE 10.4.1 SHUTTLE TANKER REQUIREMENTS

STATE	SIGNIFICANT WAVE HEIGHT (FT)
1	$H_S < 8$
2	$8 < H_S < 14$
3	$14 < H_S < 26$
4	$H_S > 26$

TABLE 10.4.2 WAVE STATES FOR SEASTATE MODEL

PRESENT WAVE STATE	NEXT WAVE STATE 1	2	3	4
1	0.97660	1.00000	1.00000	1.00000
2	0.16900	0.96530	1.00000	1.00000
3	0.00000	0.25810	1.00000	1.00000
4	0.00000	0.00000	1.00000	1.00000

TABLE 10.4.3 WAVE STATE TRANSITION MATRIX, APRIL TO SEPTEMBER, IDEALIZED MODEL

PRESENT WAVE STATE	NEXT WAVE STATE 1	2	3	4
1	0.91370	0.99970	1.00000	1.00000
2	0.10160	0.93520	1.00000	1.00000
3	0.00000	0.19170	0.98190	1.00000
4	0.00000	0.00000	0.41180	1.00000

TABLE 10.4.4 WAVE STATE TRANSITION MATRIX, OCTOBER TO MARCH, IDEALIZED MODEL

SENSITIVITY VARIABLE	PRODUCTION RATE	PRODUCTION EFFICIENCY (%) 50,000 BPD	100,000 BPD
Mooring	6 ft	98.1	89.9
Wave	8 ft*	99.2	97.0
Height	10 ft	99.2	98.8
Buffer	500 MBBL	95.9	88.1
Storage	960 MBBL*	99.2	97.0
Capacity	1,500 MBBL	99.3	98.7
Shuttle	6 Hrs	99.2	97.2
Tanker	15 Hrs*	99.2	97.0
Loading	24 Hrs	99.2	95.6
Time			
No. of	5*	99.2	97.0
Shuttle	4	99.2	96.8
Tankers on	3	99.2	90.4
Seattle			
Trade Route			
No. of	4*	99.2	97.0
Shuttle	3	99.2	96.5
Tankers on	2	99.2	82.0
Valdez			
Trade Route			

Note: Only one variable altered at a time and a * denotes the base case values.

TABLE 10.4.5 SENSITIVITY STUDY RESULTS

ITEM	100 MBOPD		50 MBOPD	
WATER DEPTH	300 FT	150 FT	300 FT	150 FT
1. Engineering/Management				
Marine Insurance (10%)	0.7	0.6	0.6	0.6
2. Mooring System				
Materials & Fabrication	0.4	0.4	0.4	0.4
Transportation	0.6	0.6	0.6	0.6
Installation	1.6	1.6	1.6	1.6
3. Riser/Riser Base				
Materials & Fabrication	2.5	1.9	2.1	1.8
Transportation	0.6	0.6	0.6	0.6
Installation	0.6	0.6	0.6	0.6
4. Test & Commission	0.2	0.2	0.2	0.2
TOTAL	7.2	6.5	6.7	6.2

TABLE 10.5.1 REMOTE LODING BUOY COST SENSITIVITY ($MM)

Engineering/Management (10%)	0.3
Modifications	0.3
Outfitting	2.3
Shipyard Costs	0.3
Total	3.2

TABLE 10.5.2 ESTIMATED COSTS TO MODIFY STANDARD 60,000 DWT SHUTTLE TANKERS FOR APPLICATION WITH THE REMOTE LOADING BUOY

PIPE DIAMETER (INCH)	WEIGHT COAT (INCH)
>14	1.5
14-24	2.0
26-30	2.5
32-34	3.0
36-40	3.5

TABLE 11.1.1 TYPICAL WEIGHT COATING REQUIREMENTS AS A FUNCTION OF PIPE DIAMETER FOR MARINE PIPELINES

Water Depth	300 Ft
Production Rate	100,000 bpd

ITEM	COST $MM	NOTES
Jacket	256.0	Table 5.3.1
Marine Pipeline	143.0	Figures 11.1.3 and 11.2.3 4 users assumed each producing 100,000 bopd, 32 inch diameter pipe for 170 miles.
Total	399.0	

TABLE 12.0.1 ESTIMATED COST FOR SCENARIO 1

Water Depth	300 Ft
Production Rate	100,000 bpd

ITEM	COST ($MM)	NOTES
Jacket	256.0	Table 5.3.1
Pipeline	12.0	Figure 11.2.1, 1 Mile @ 25" diameter
FSO	98.8	Table 9.3.6
Total	366.8	

TABLE 12.0.2 <u>ESTIMATED COST FOR SCENARIO 2</u>

Water Depth	150 Ft
Production Rate	100,000 bpd

ITEM	COST ($MM)	NOTES
Jacket	333.5	Table 5.3.7, 4 Multiple Jackets
Marine Pipeline	29.0	Figures 11.1.3 and 11.2.3, 4 users assumed each producing 100,000 bopd, 26 inch diameter pipe for 40 miles.
Total	362.5	

TABLE 12.0.3 ESTIMATED COST FOR SCENARIO 3

Water Depth		300 Ft
Production Rate		100,000 bpd

ITEM	COST ($MM)	NOTES
Hybrid Platform	414.7	Table 6.3.3
Pipeline	23.0	Figure 11.2.2, 1 Mile
Loading Buoy	7.2	Table 10.5.1
Total	444.9	

TABLE 12.0.4 ESTIMATED COSTS FOR SCENARIO 4

Water Depth	150 Ft
Production Rate	100,000 bpd

ITEM	COST ($MM)	NOTES
Hybrid Platform	373.3	Table 6.3.2
Pipeline	23.0	Figure 11.2.2, 1 Mile
Loading Buoy	6.5	Table 10.5.1
Total	402.8	

TABLE 12.0.5 ESTIMATED COST FOR SCENARIO 5

Water Depth		300 Ft
Production Rate		100,000 bpd

ITEM	COST ($MM)	NOTES
Concrete Gravity Platform	392.5	Table 7.4.2
Pipeline	23.0	Figure 11.2.2, 1 Mile
Loading Buoy	7.2	Table 10.5.1
Total	422.7	

TABLE 12.0.6 ESTIMATED COST FOR SCENARIO 6

Water Depth		150 Ft
Production Rate		100,000 bpd

ITEM	COST ($MM)	NOTES
Concrete Gravity Platform	359.5	Table 7.4.1
Pipeline	23.0	Figure 11.2.2, 1 Mile
Loading Buoy	6.5	Table 10.5.1
Total	389.0	

TABLE 12.0.7 ESTIMATED COST FOR SCENARIO 7

Water Depth	250 ft
Production Rate	100,000 bpd

ITEM	COST ($MM)	NOTES
*FPSO + Templates	178.6	Table 9.3.1
Pipelines, Control Cable Installation	21.1	Appendix A, 8 x 2 Mile
Total	199.7	

* Floating systems have no provisions for drilling costs.

TABLE 12.0.8 ESTIMATED COSTS FOR SCENARIO 8

Scenario	Platform	Pipelines	FPSO + Template	FSO	Loading Buoy	Total
1	220.4	115.0	-	-	-	335.4
2	220.4	11.0	-	98.0	-	329.4
3	297.9	20.0	-	-	-	317.9
4	324.8	23.0	-	-	6.7	354.5
5	287.5	23.0	-	-	6.2	316.7
6	319.2	23.0	-	-	6.7	348.9
7	293.4	23.0	-	-	6.2	322.6
8	-	21.1	163.4*	-	-	184.5

* FPSO system includes no drilling costs.

TABLE 12.0.9 ESTIMATED COSTS ($MM) OF SCENARIOS 1 - 8 FOR 50,000 BPD PRODUCTION FABRICATION IN JAPAN

Scenario	Platform	Pipelines	FPSO + Template	FSO	Loading Buoy	Total
1	256.0	143.0	-	-	-	399.0
2	256.0	12.0	-	98.8	-	366.8
3	333.5	29.0	-	-	-	362.5
4	414.7	23.0	-	-	7.2	444.9
5	373.3	23.0	-	-	6.5	402.8
6	392.5	23.0	-	-	7.2	422.7
7	359.5	23.0	-	-	6.5	389.0
8	-	21.1	178.6*	-	-	199.7

* FPSO system includes no drilling costs.
All floating system fabrication in Japan.

TABLE 12.0.10 ESTIMATED COSTS ($MM) OF SCENARIOS 1 - 8 FOR 100,000 BPD PRODUCTION FIXED PLATFORM FABRICATION IN JAPAN

Scenario	Platform	Pipelines	FPSO + Template	FSO	Loading Buoy	Total
1	302.0	143.0	-	-	-	445.0
2	302.0	12.0	-	98.8	-	412.8
3	343.6	29.0	-	-	-	372.6
4	483.2	23.0	-	-	7.2	513.4
5	433.1	23.0	-	-	6.5	462.6
6	458.7	23.0	-	-	7.2	488.9
7	416.7	23.0	-	-	6.5	446.2
8	-	21.1	178.6	-	-	199.7

* No drilling costs included for FPSO.
All floating system fabrication in Japan.

TABLE 12.0.11 <u>ESTIMATED COSTS ($MM) OF SCENARIOS 1 - 8 FOR 100,000 BPD PRODUCTION FIXED PLATFORM FABRICATION IN U.S WEST COAST</u>

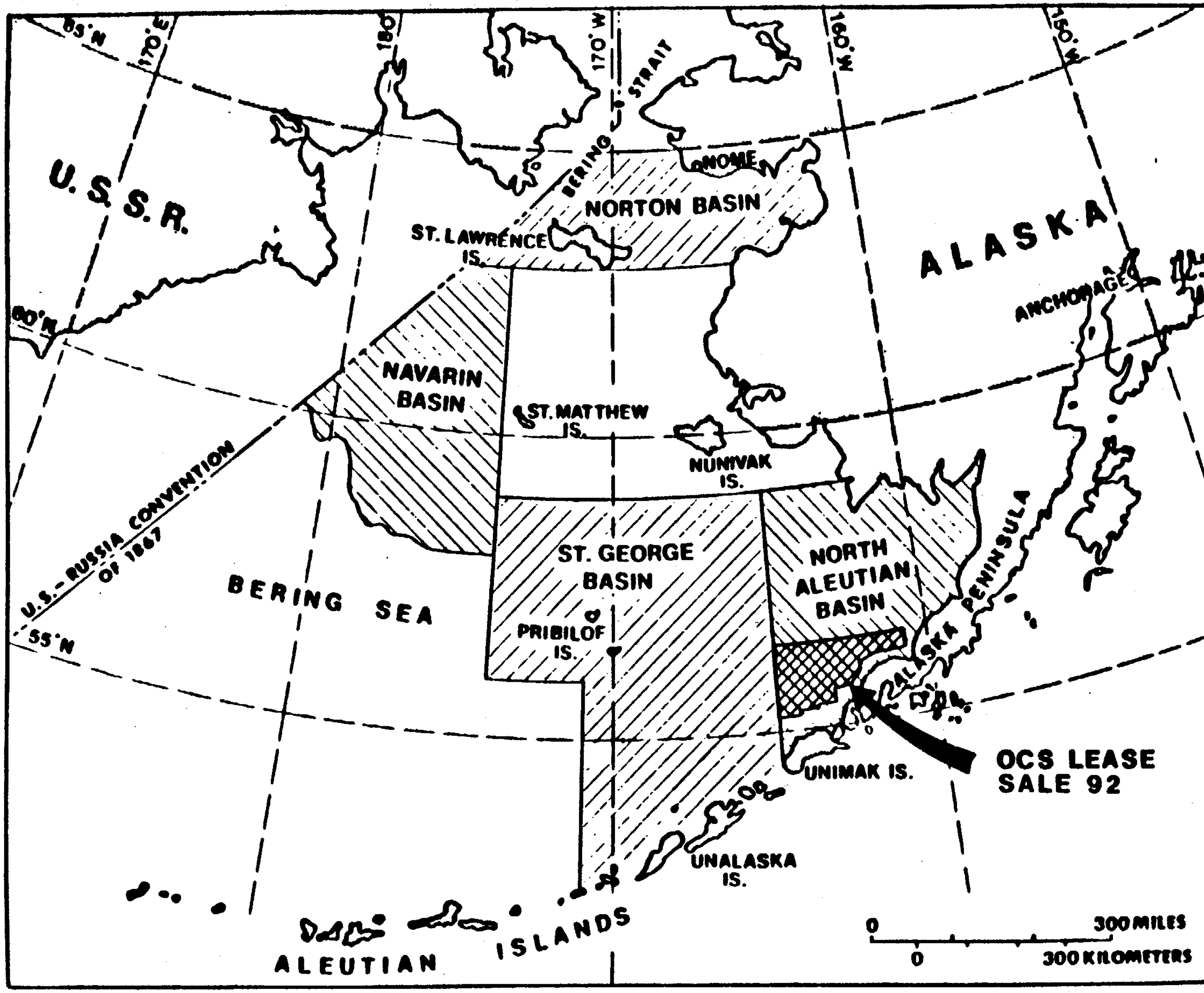


FIGURE 1.0.1 NORTH ALEUTIAN LEASE SALE AREA

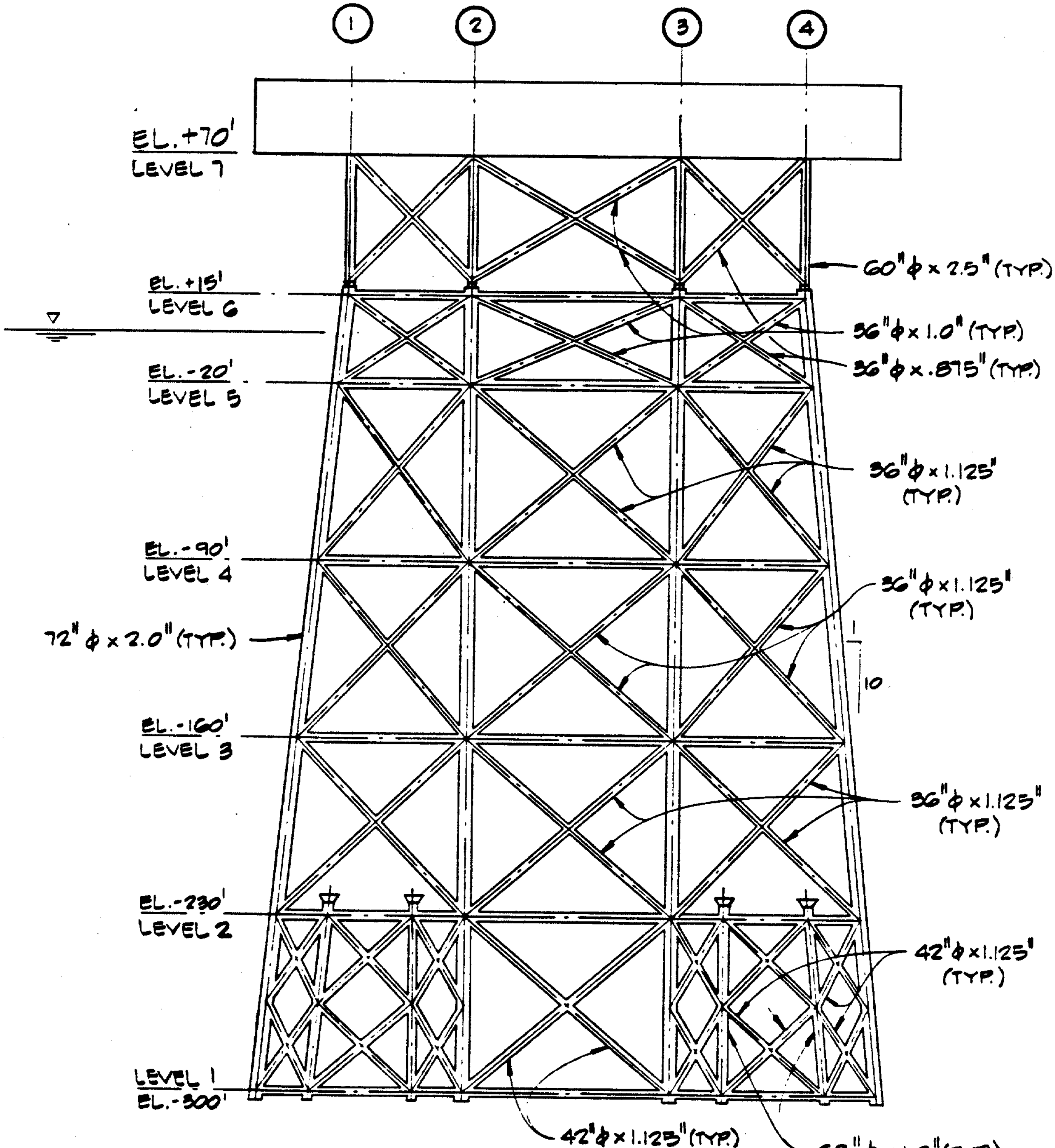


FIGURE 1.0.2 PILED JACKET CONCEPT

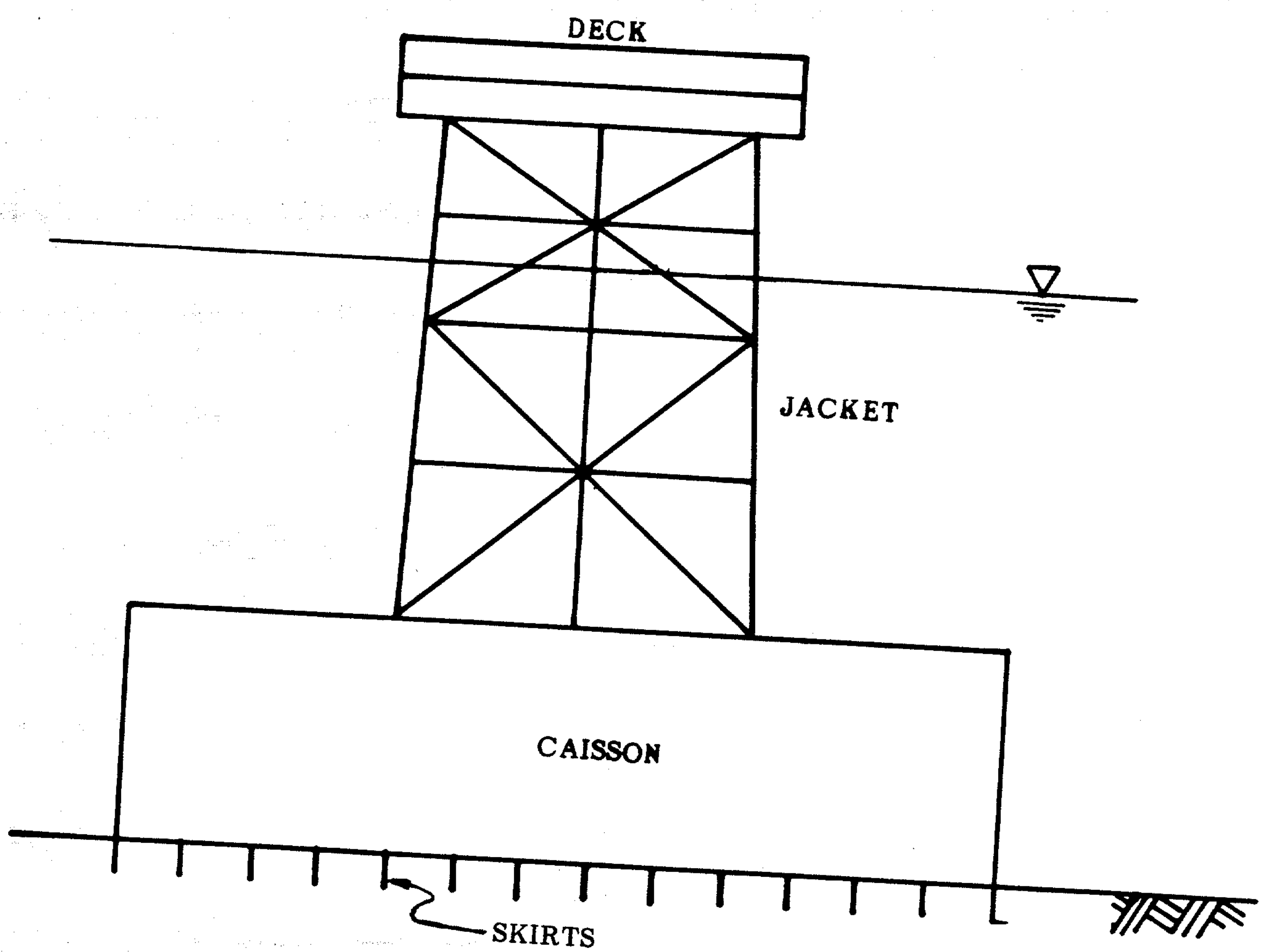
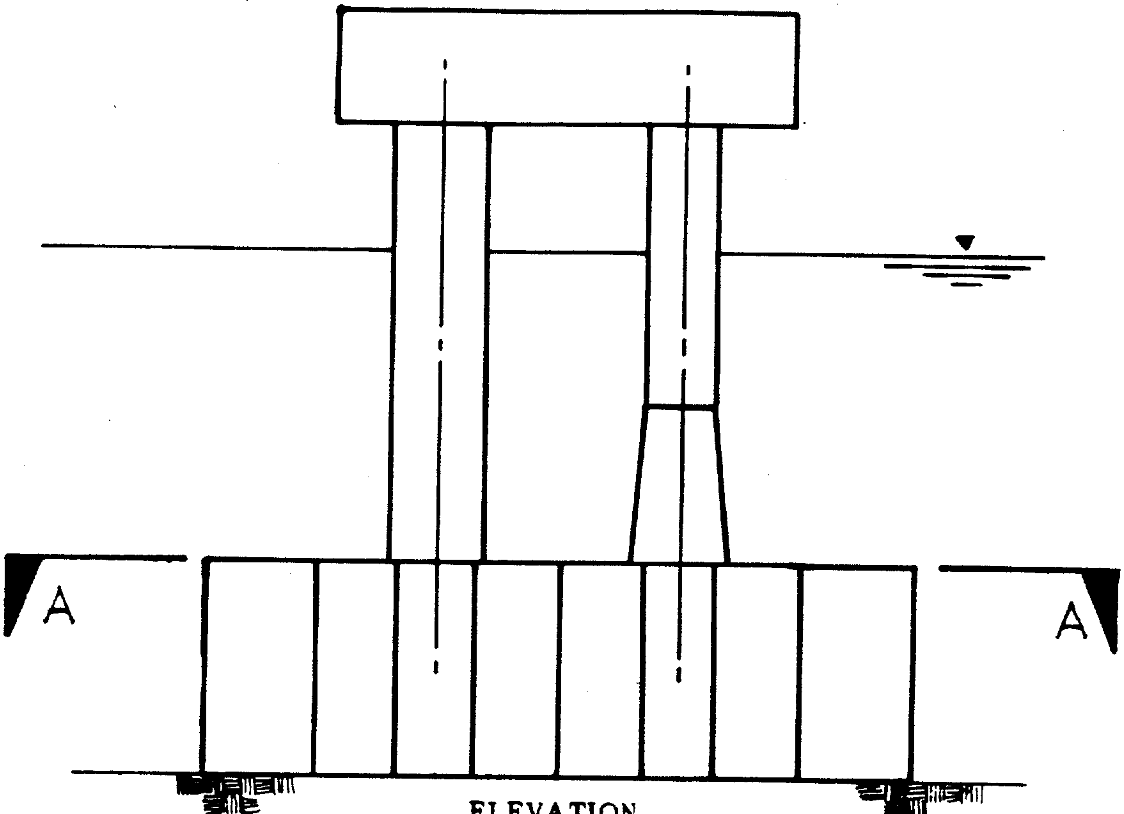
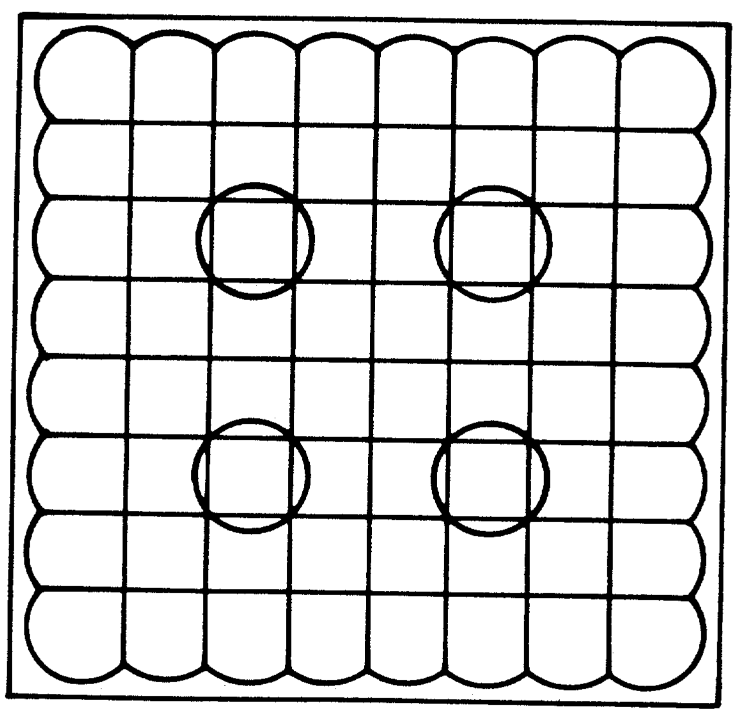


FIGURE 1.0.3 HYBRID PLATFORM



ELEVATION



SECTION "A-A"

FIGURE 1.0.4 CONCRETE GRAVITY PLATFORM

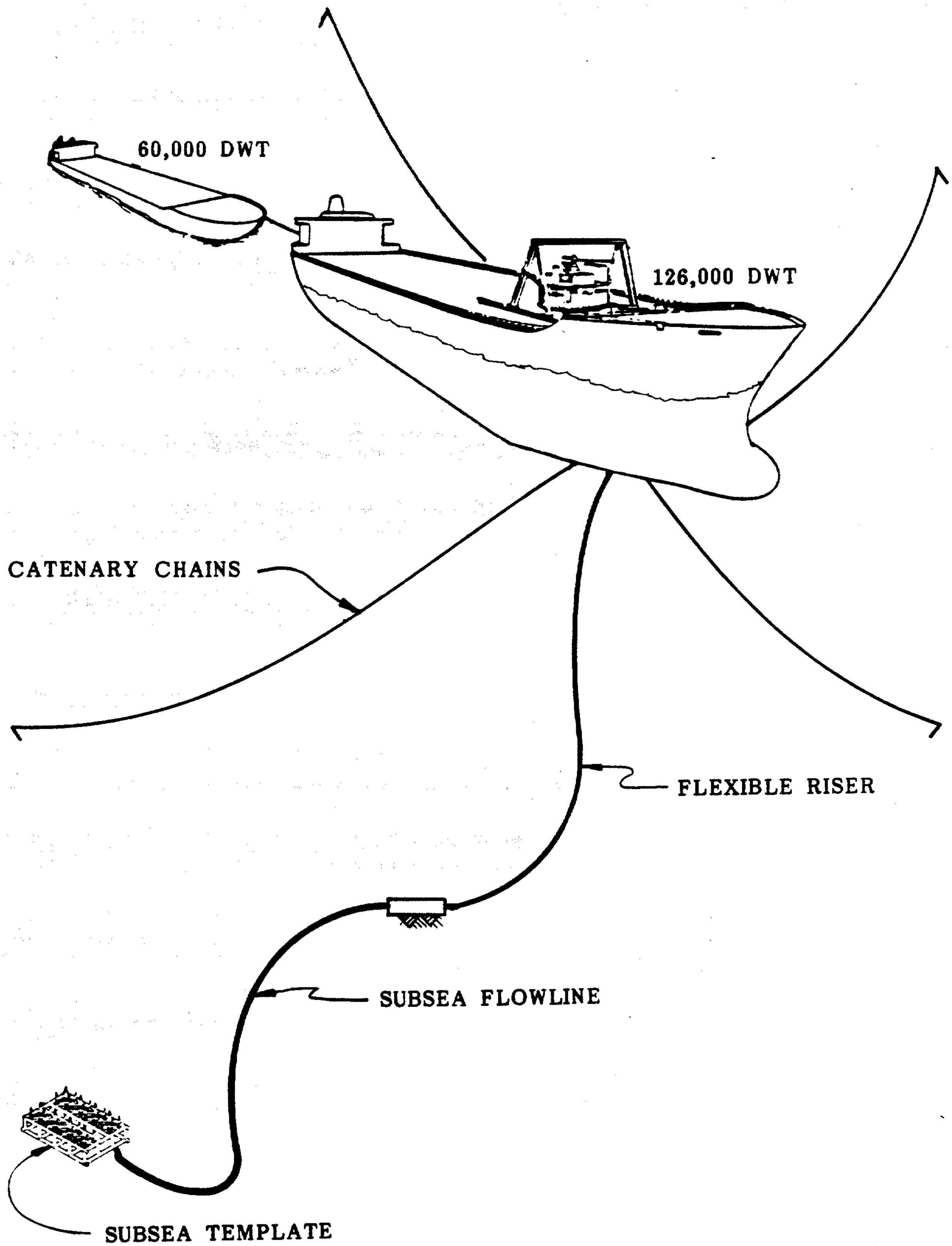


FIGURE 1.0.5 FLOATING SYSTEMS

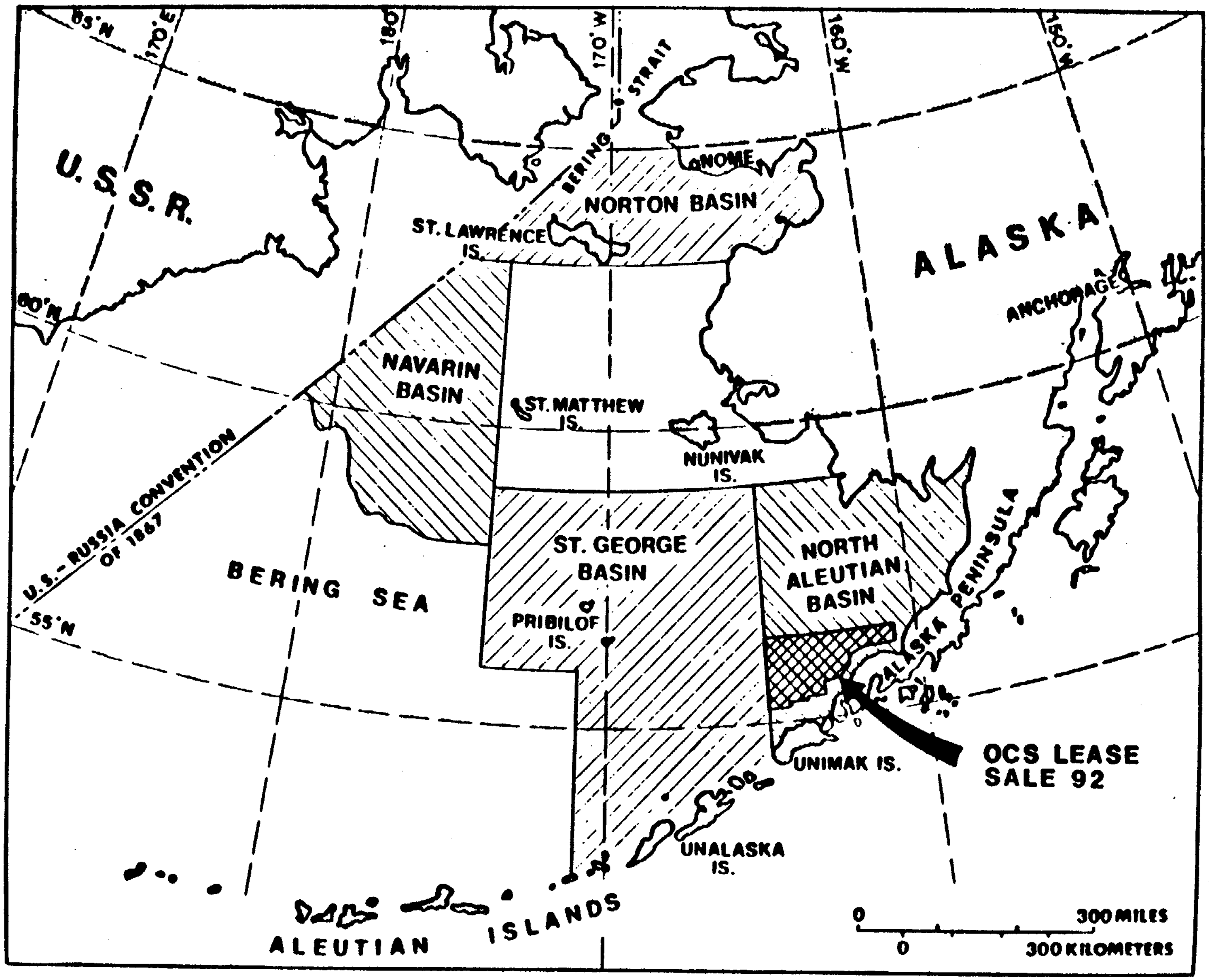


FIGURE 2.0.1 NORTH ALEUTIAN BASIN LEASE SALE AREA

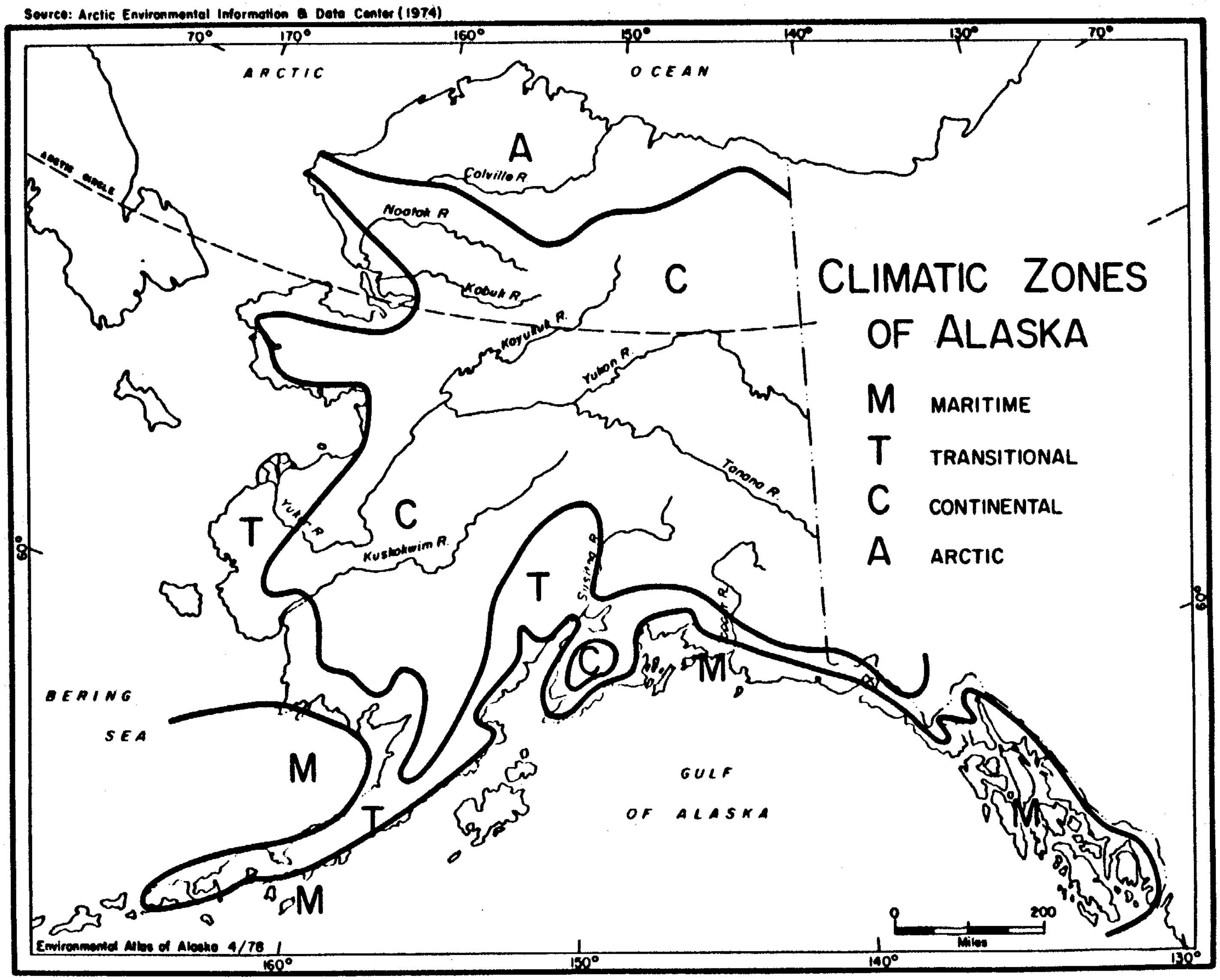


PLATE 23

FIGURE 2.1.1 ALASKAN CLIMATIC ZONES

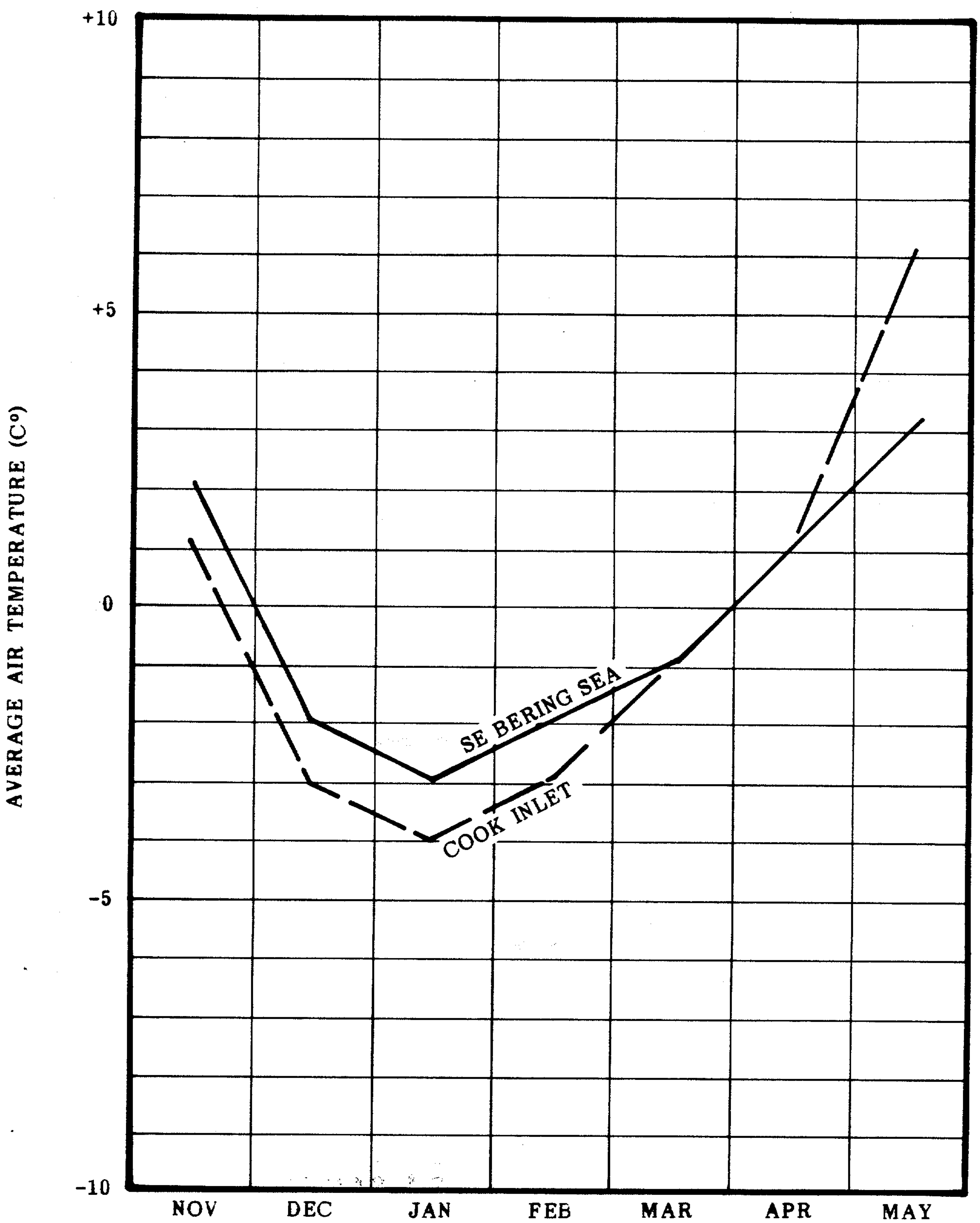


FIGURE 2.1.2 AVERAGE AIR TEMPERATURE IN THE LEASE SALE AREA

Visibility obstructions

F = Fog

10 St. Paul

	F
Jan	17.5
Feb	17.8
Mar	16.2
Apr	18.5
May	30.0
Jun	40.8
Jul	56.1
Aug	43.6
Sep	23.8
Oct	8.5
Nov	10.0
Dec	10.9
Ann	24.6

11 Port Heiden

	F
Jan	7.1
Feb	6.8
Mar	8.2
Apr	7.6
May	6.7
Jun	11.3
Jul	20.3
Aug	23.2
Sep	8.3
Oct	5.0
Nov	6.6
Dec	5.1
Ann	9.7

12 Port Moller

	F
Jan	14.5
Feb	18.6
Mar	23.7
Apr	32.4
May	23.5
Jun	35.0
Jul	40.0
Aug	34.9
Sep	17.8
Oct	17.7
Nov	18.7
Dec	15.8
Ann	24.4

13 Driftwood Bay

	F
Jan	14.1
Feb	18.6
Mar	18.4
Apr	24.9
May	32.0
Jun	41.2
Jul	45.8
Aug	43.0
Sep	33.8
Oct	16.8
Nov	17.1
Dec	15.6
Ann	26.8

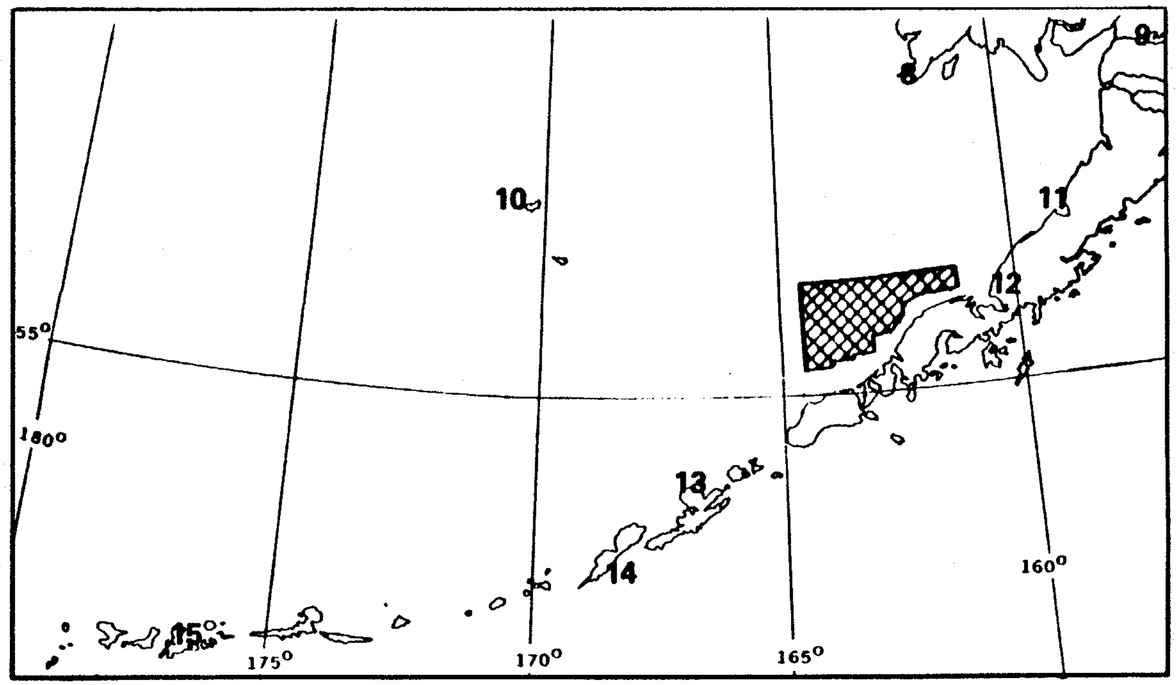


FIGURE 2.1.3 OCCURRENCES OF REDUCED VISIBILITY

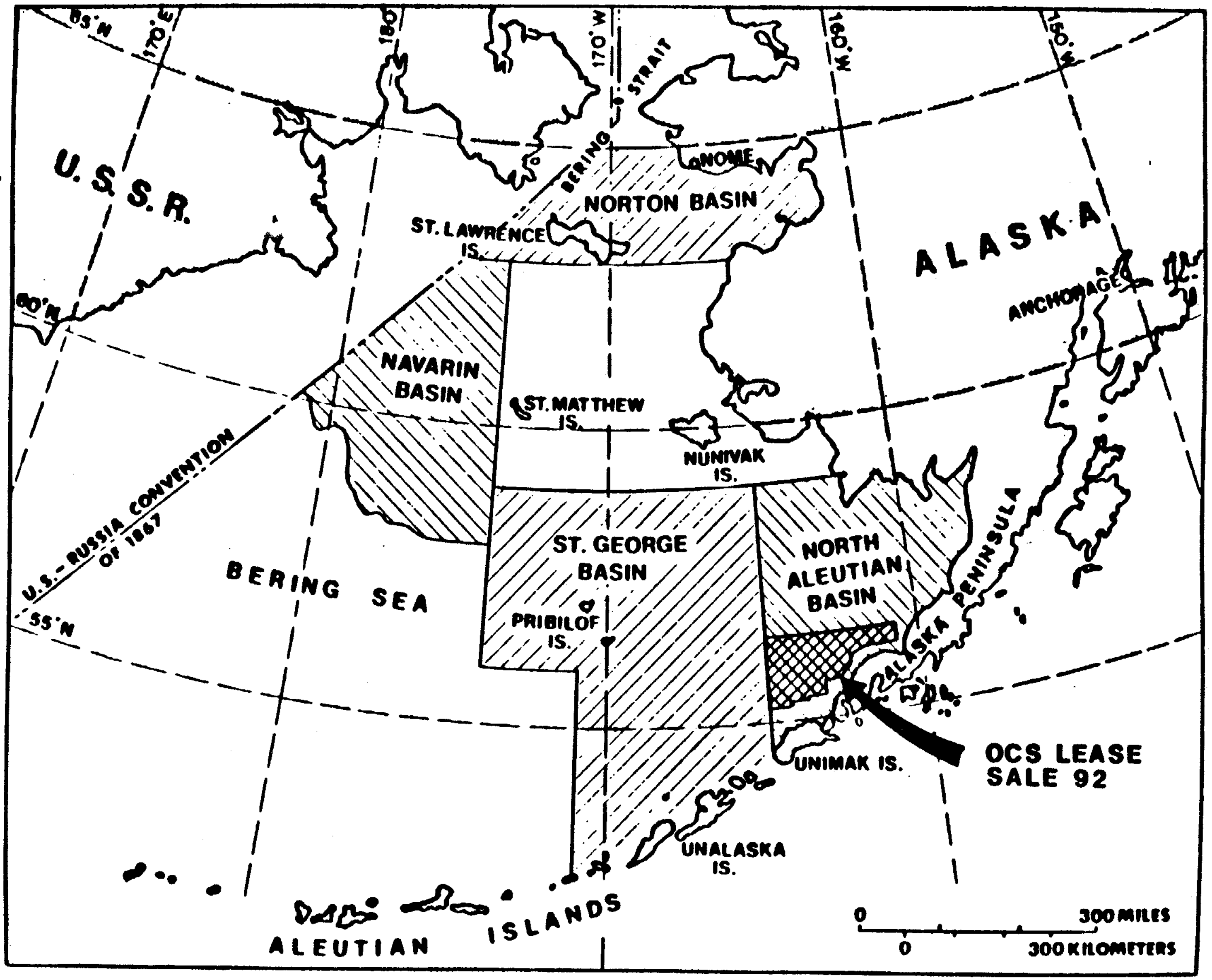


FIGURE 2.2.1 NORTH ALEUTIAN BASIN LEASE SALE AREA

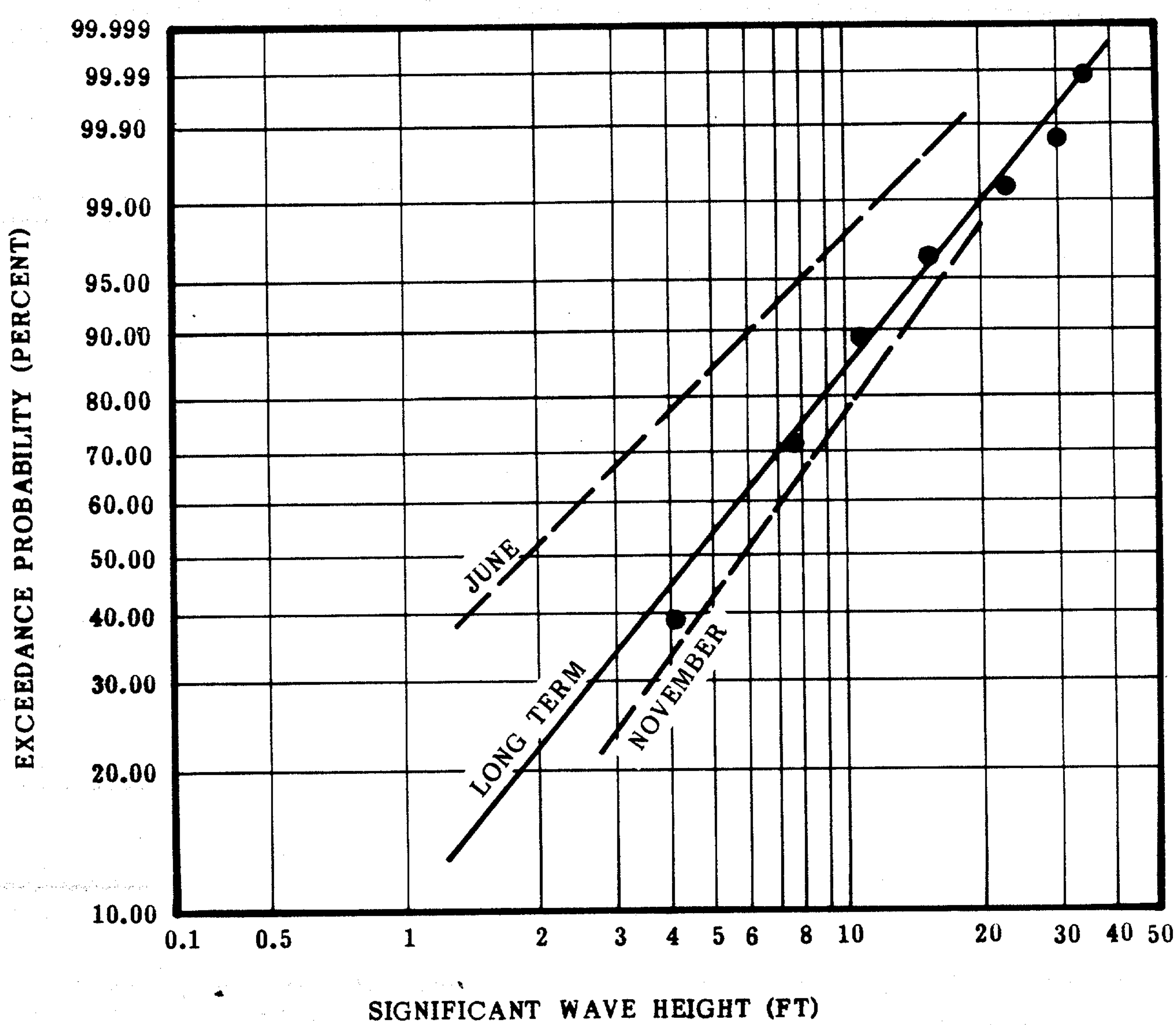


FIGURE 2.2.2 LONG TERM DISTRIBUTION OF WAVE HEIGHTS

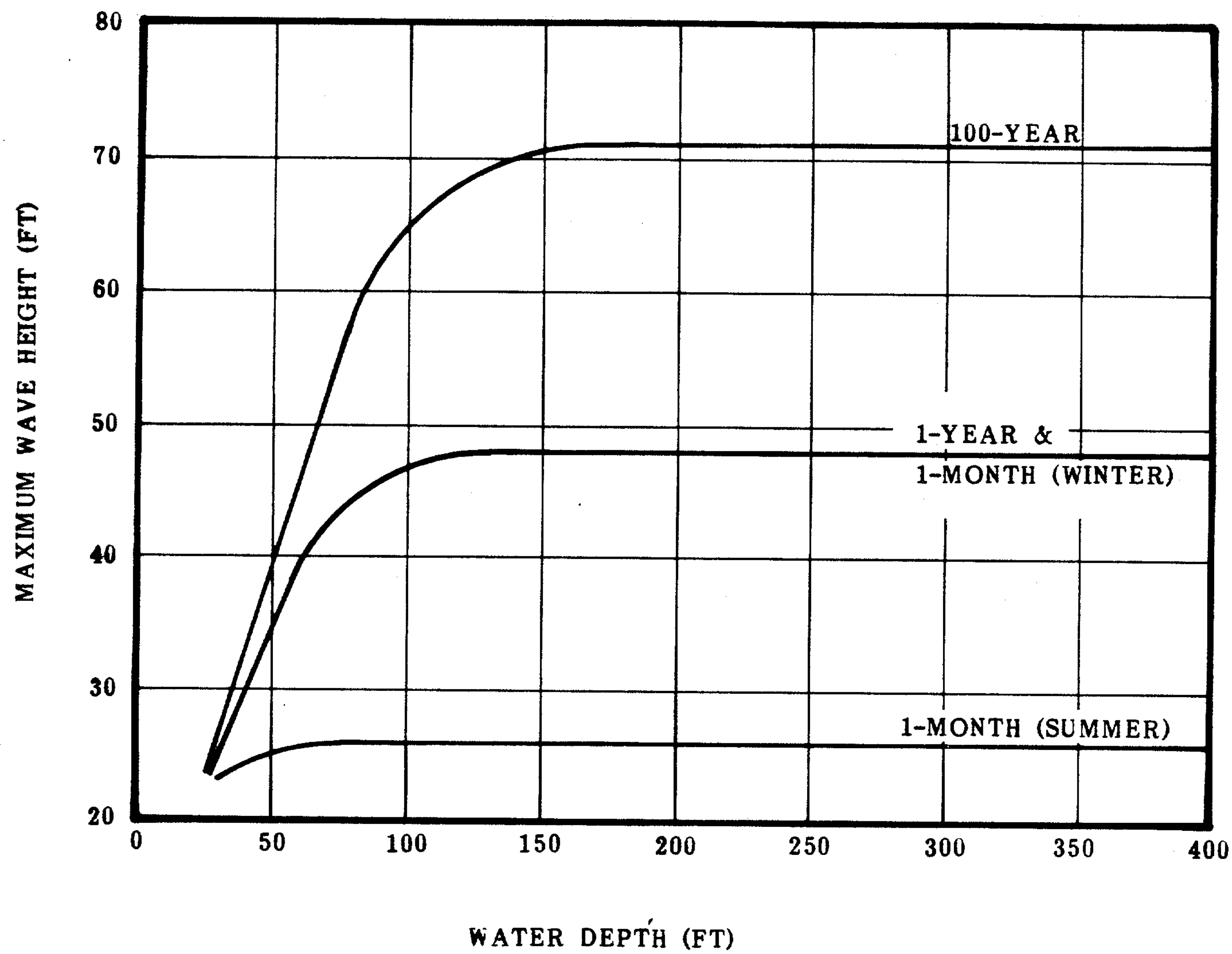


FIGURE 2.2.3 RECOMMENDED MAXIMUM DESIGN WAVE HEIGHT VS. DEPTH

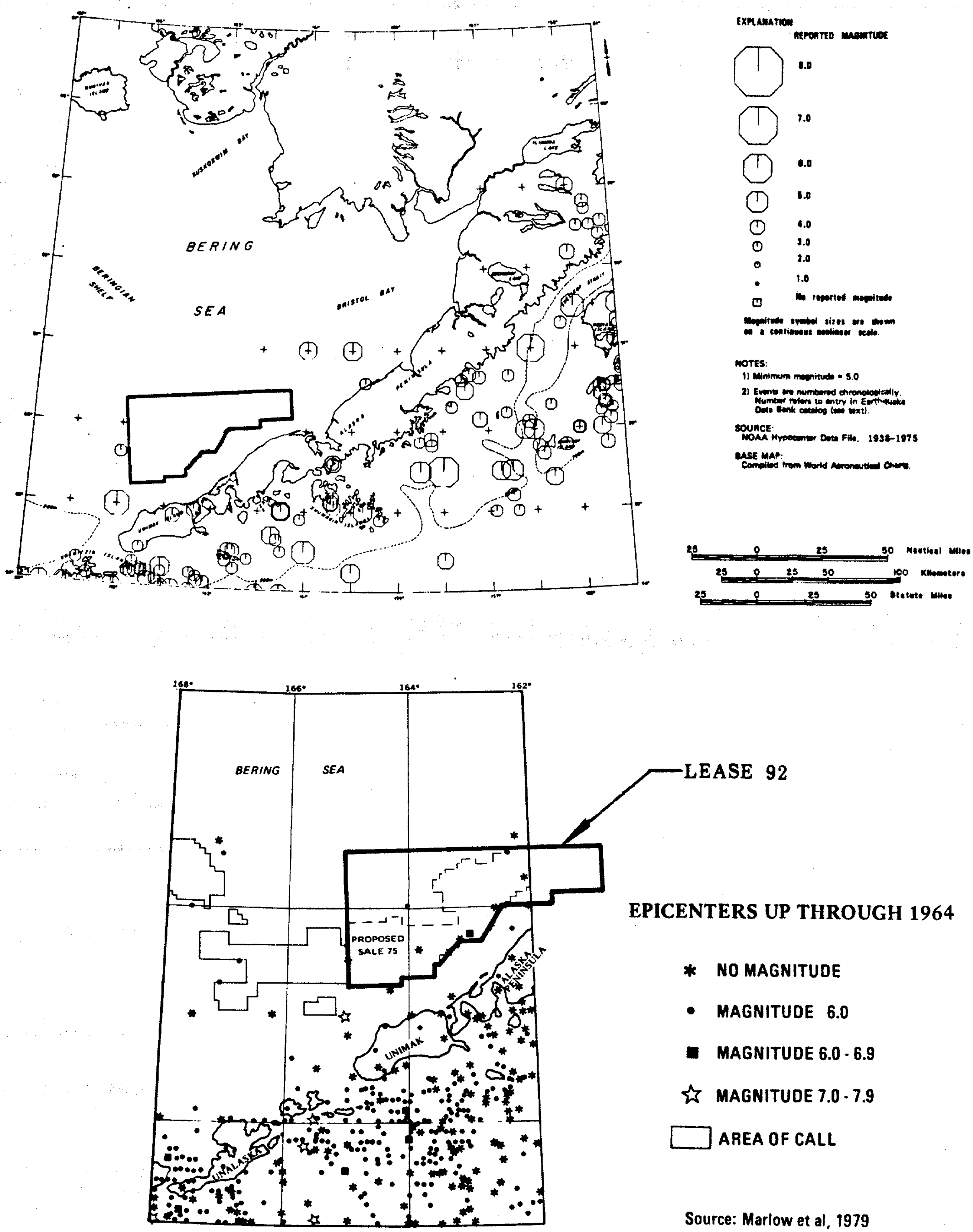


FIGURE 2.3.1 RECORDED EARTHQUAKE HISTORY (IN LEASE AREA)

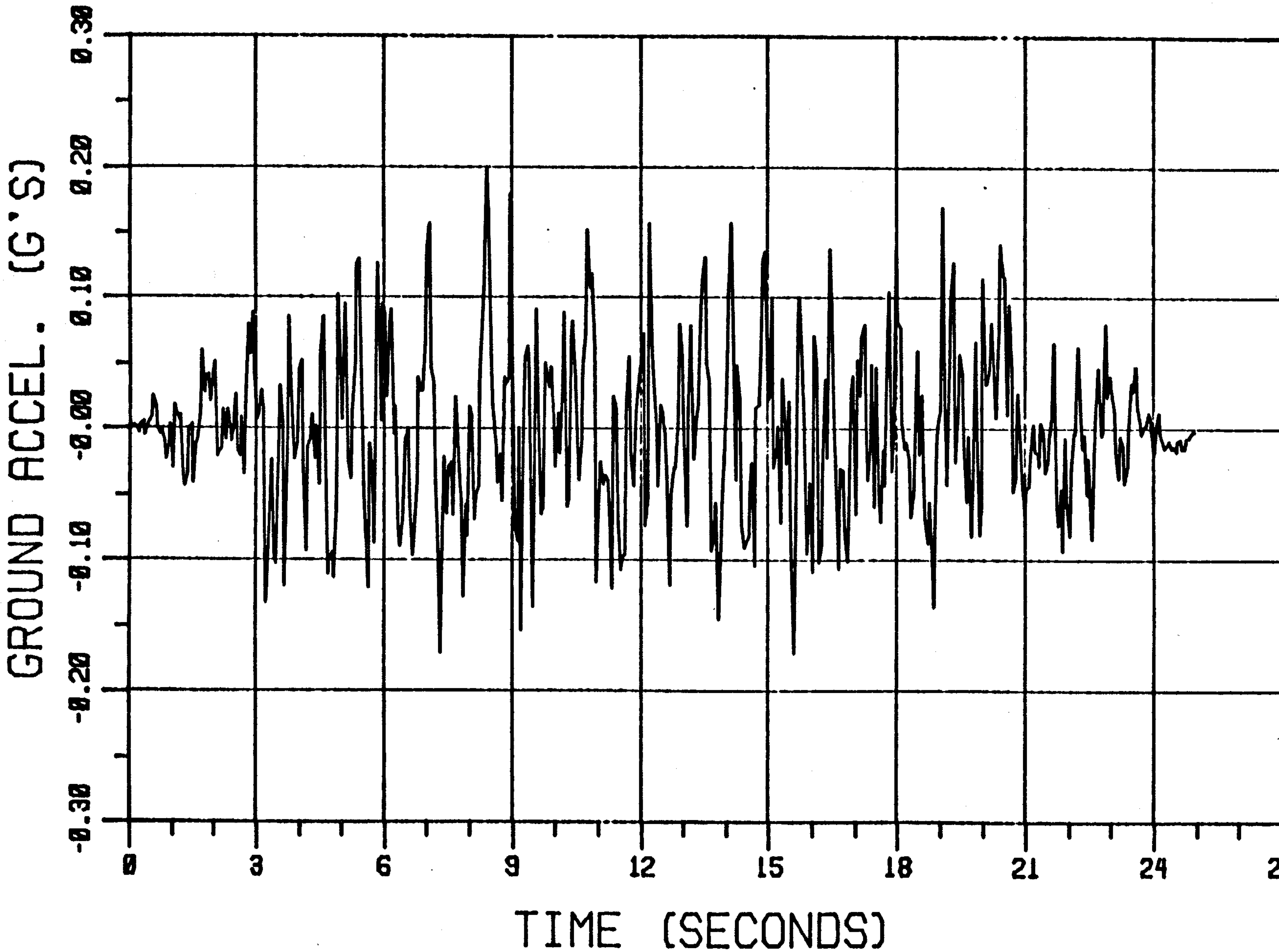


FIGURE 2.3.2 TYPICAL TIME HISTORY

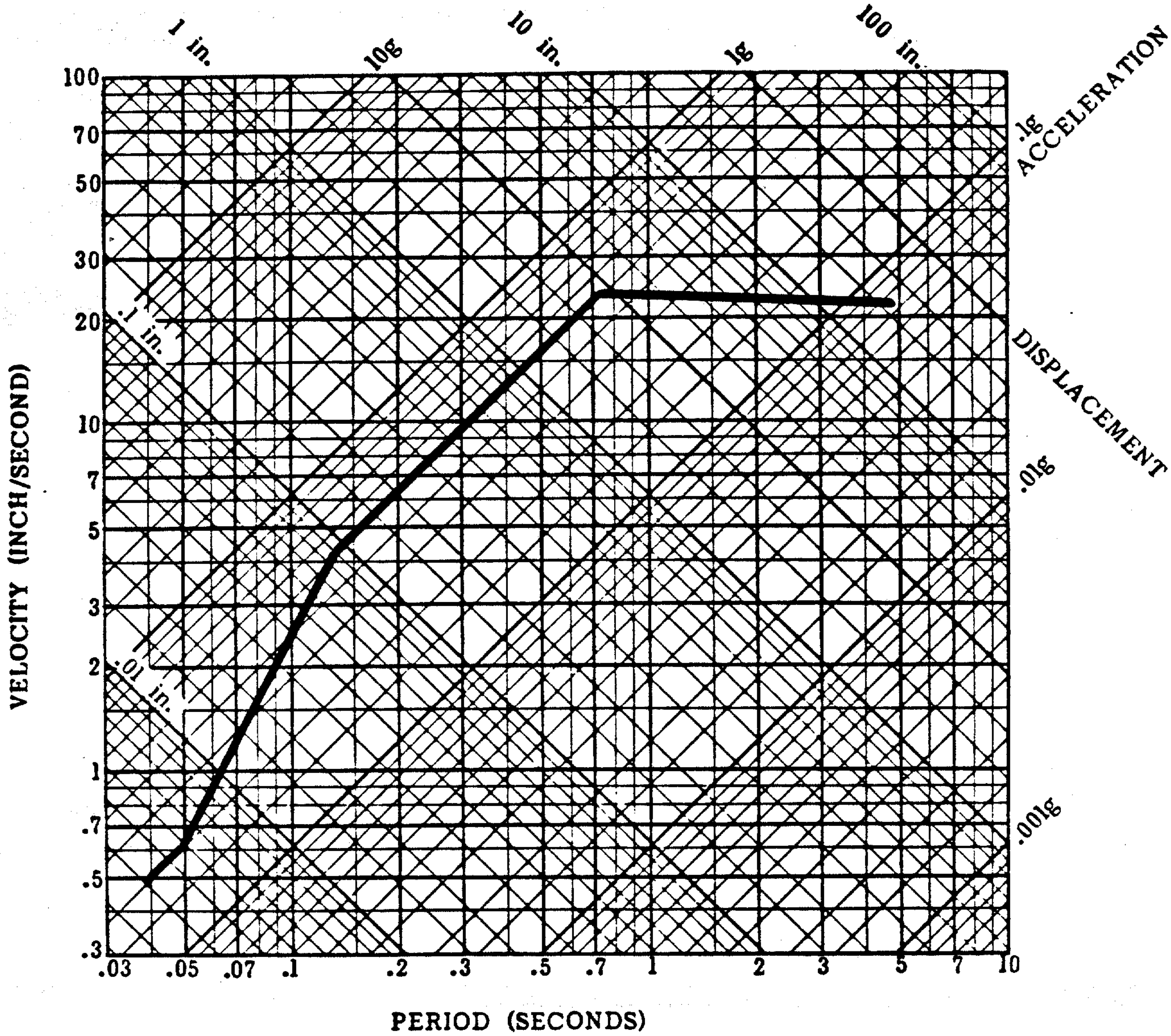


FIGURE 2.3.3 TYPCIAL RESPONSE SPECTRUM (API ZONE 3)

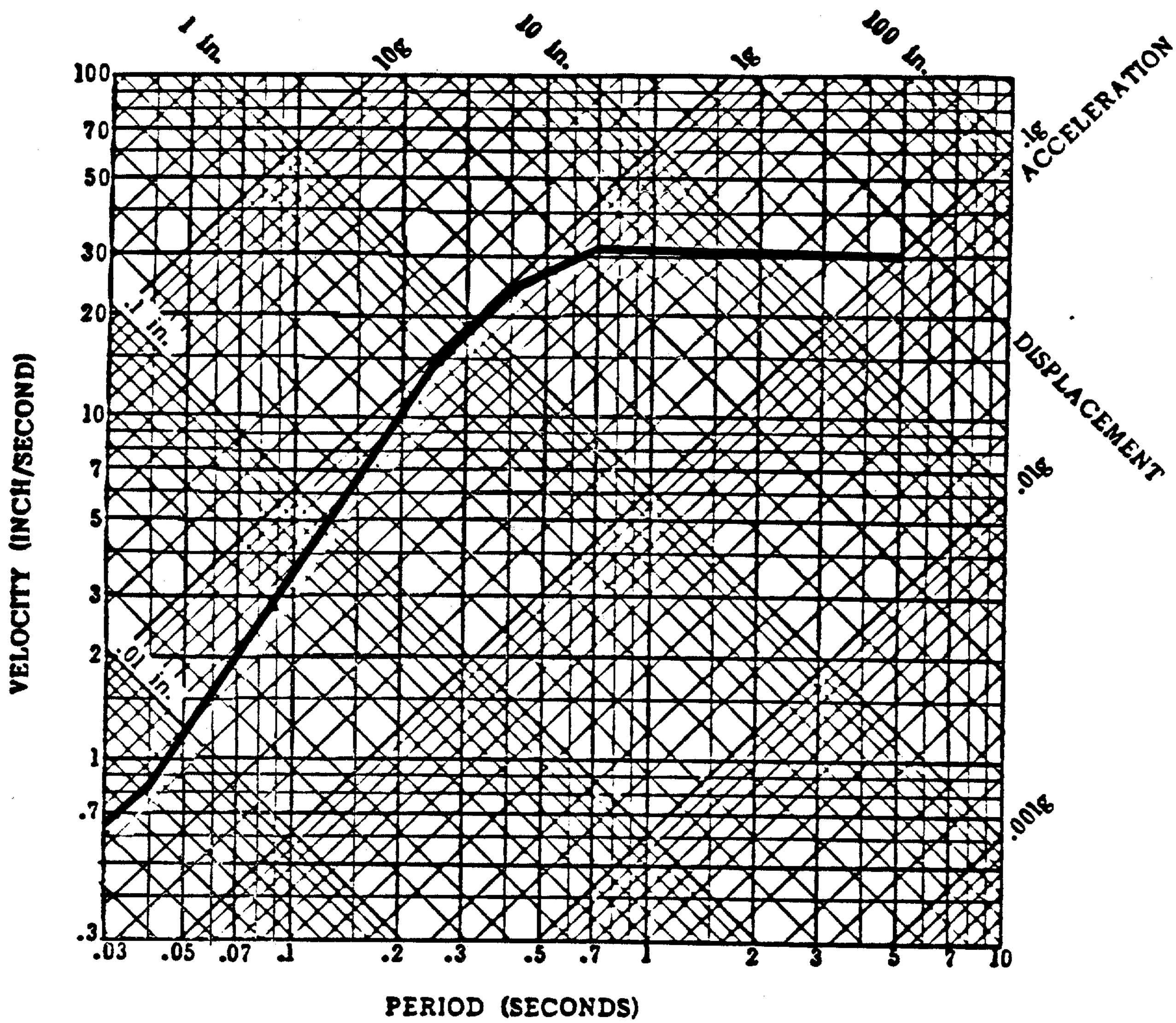
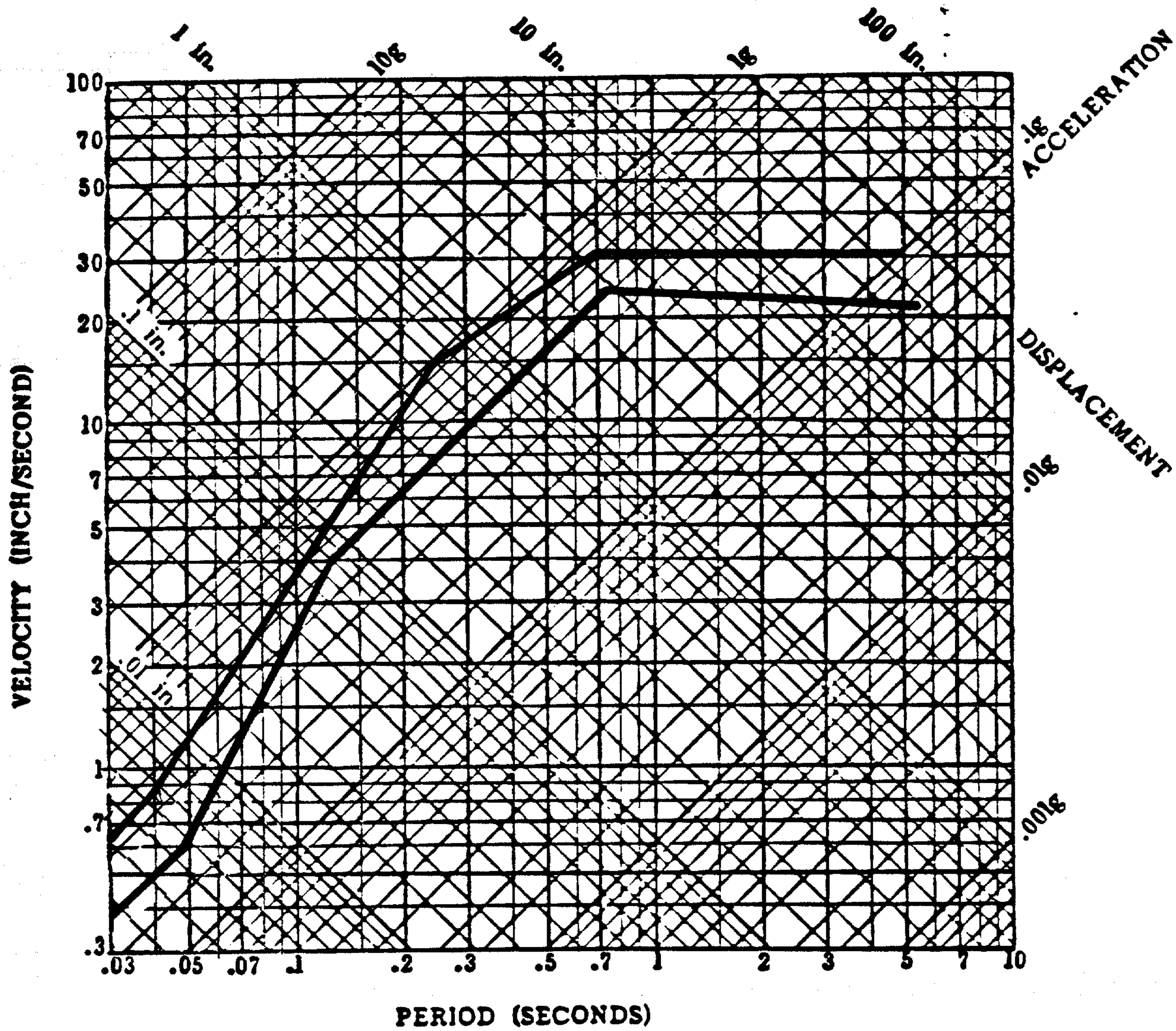


FIGURE 2.3.4 DESIGN RESPONSE SPECTRUM



API RP 2A

ZONE 3, SOIL C

PEAK G. ACCEL = 0.2g

FIGURE 2.3.5 COMPARISON OF API ZONE 3 AND DESIGN SPECTRA

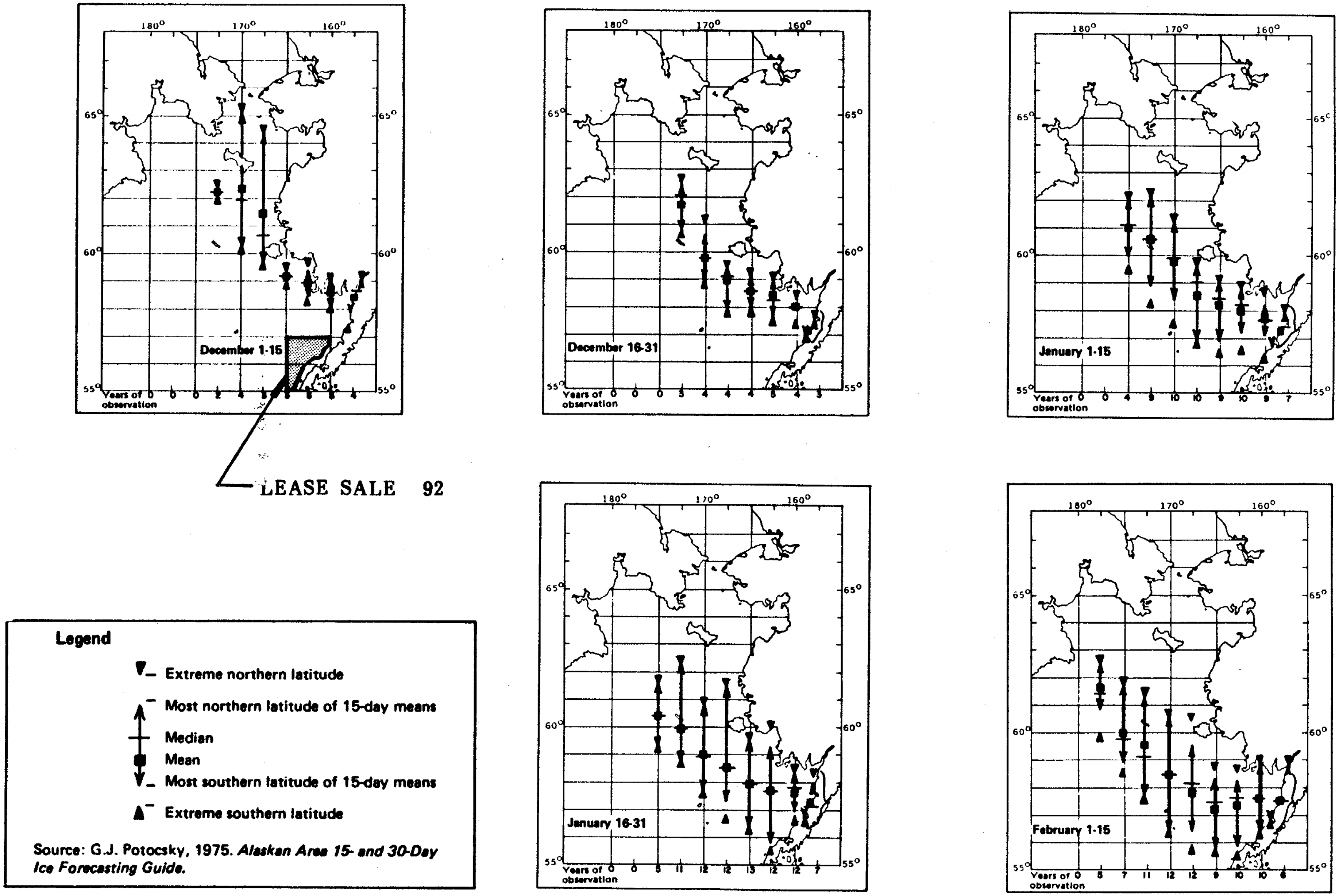


FIGURE 2.4.1 ICE OCCURRENCE AS A FUNCTION OF DATE IN THE NORTH ALEUTIAN BASIN AREA

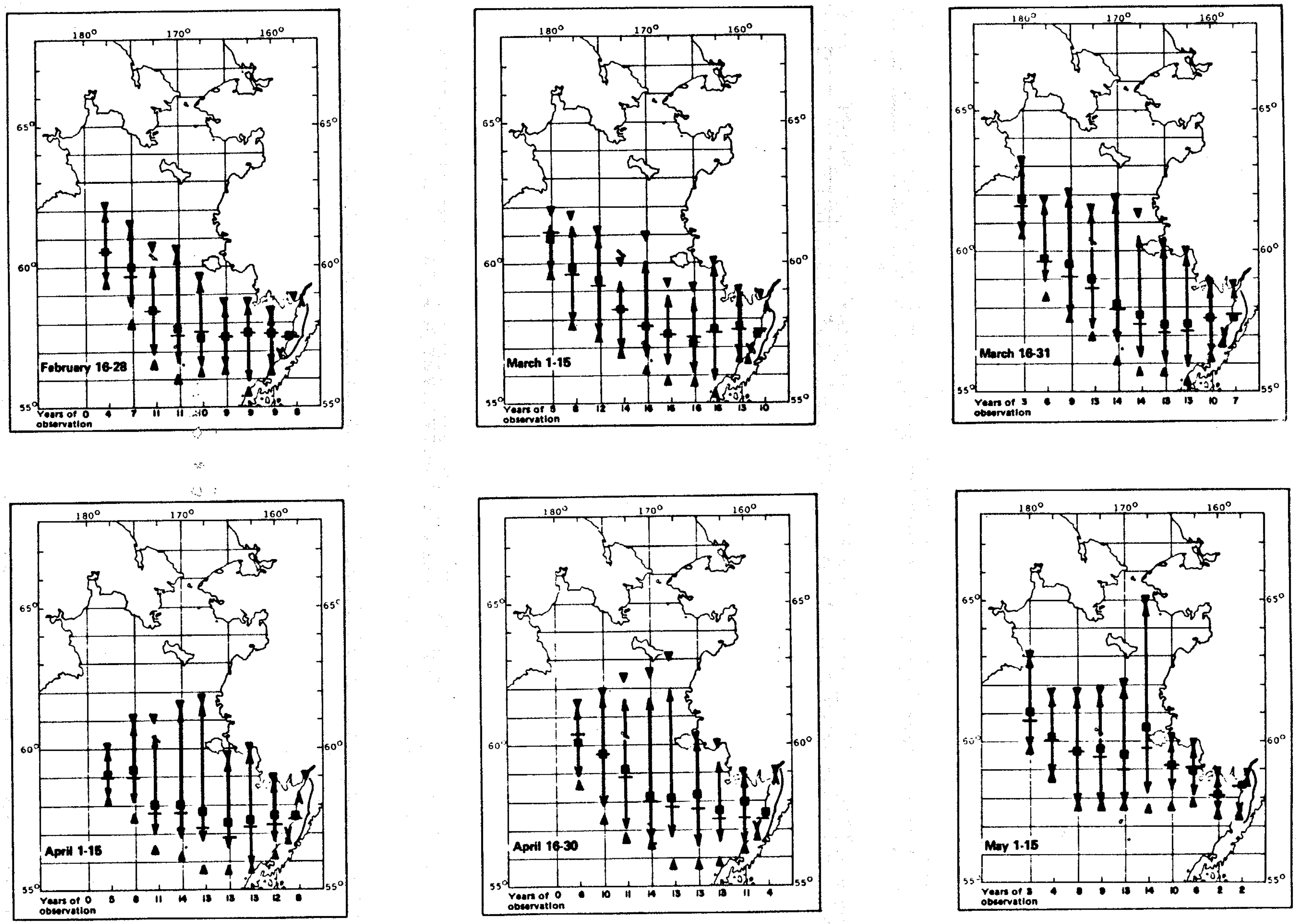


FIGURE 2.4.2 ICE OCCURRENCE AS A FUNCTION OF DATE IN THE NORTH ALEUTIAN BASIN AREA

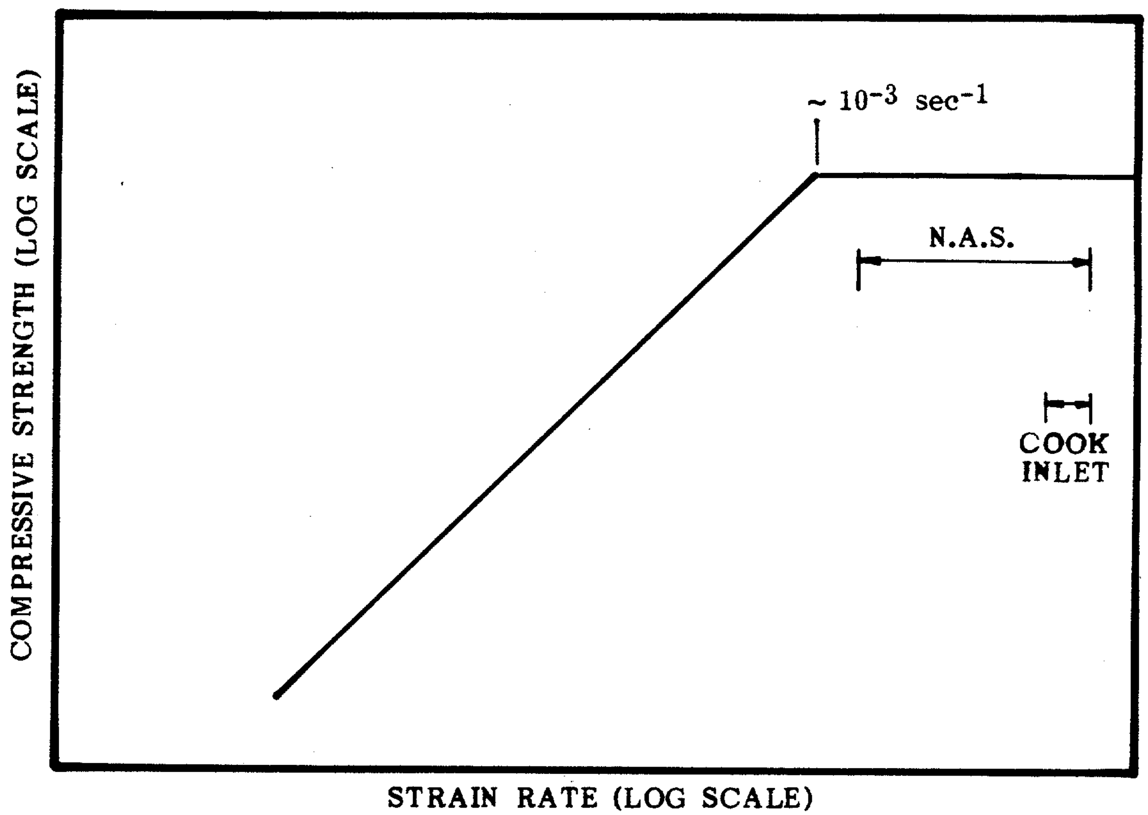


FIGURE 2.4.3 TYPICAL STRAIN-RATE DEPENDENCE OF ICE COMPRESSIVE STRENGTH

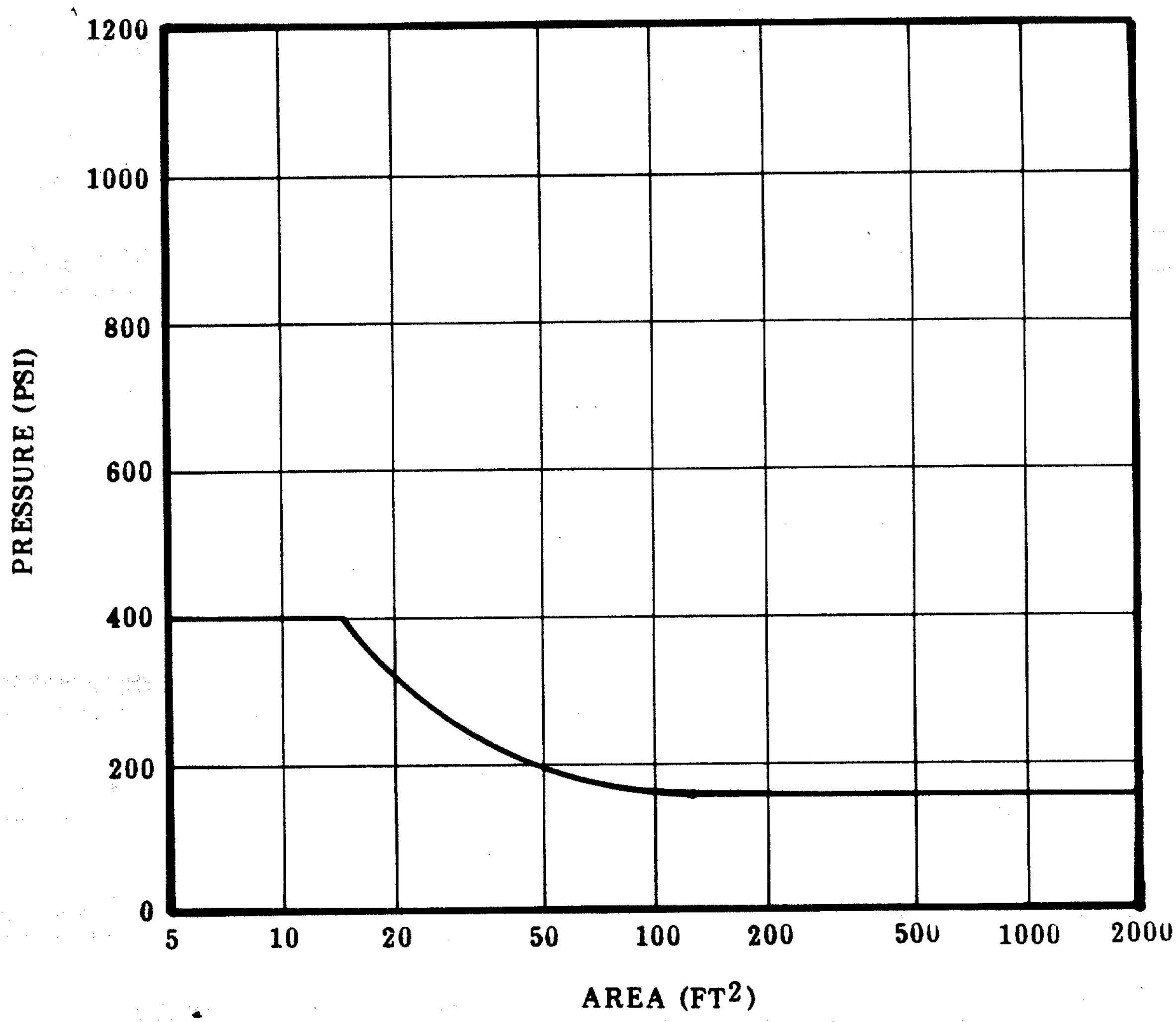


FIGURE 2.4.4 PRESSURE-AREA RELATIONSHIP FOR LOCAL DESIGN

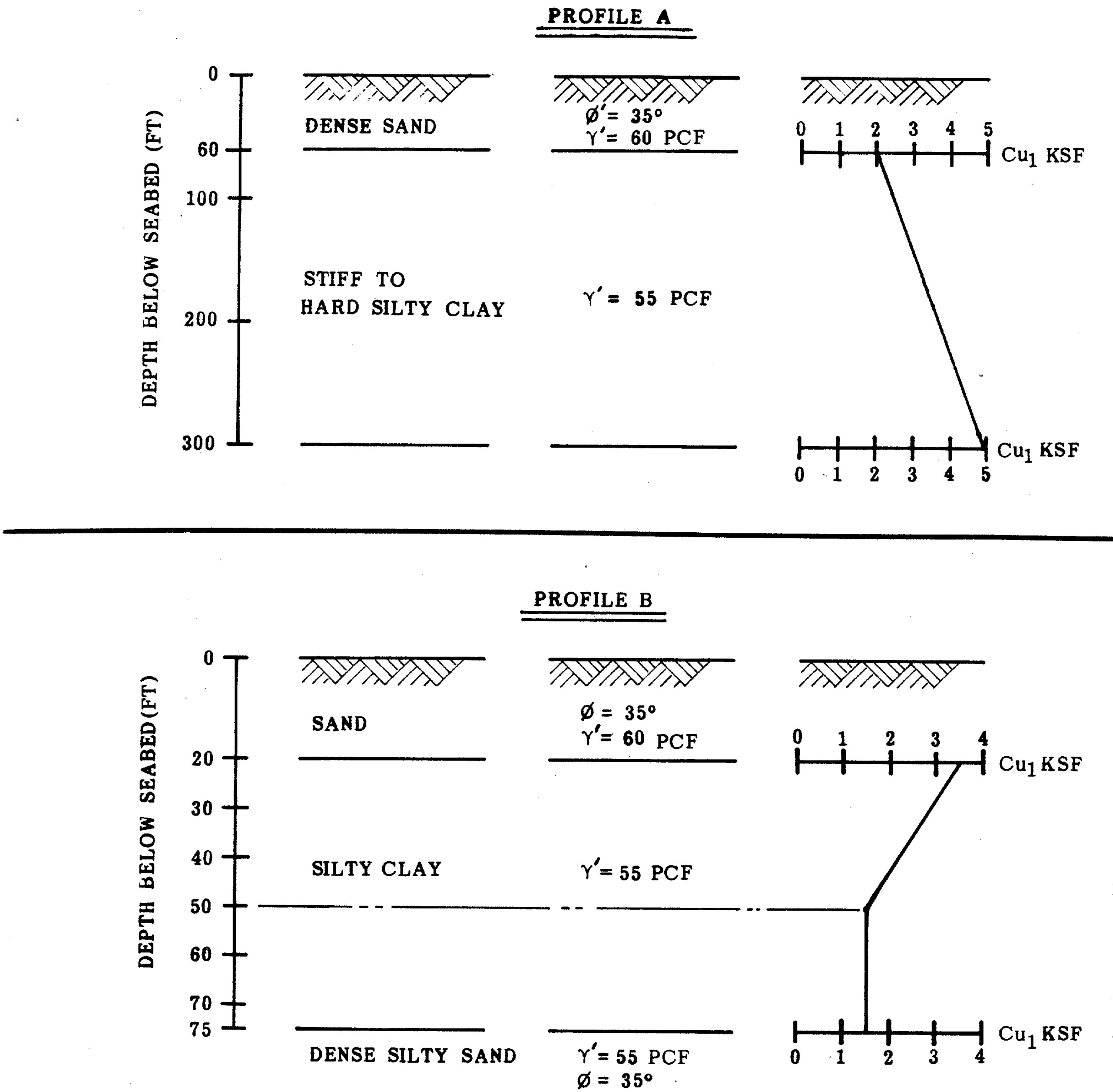


FIGURE 2.5.1 PROPOSED SOIL PROFILES FOR NORTH ALEUTIAN BASIN LEASE SALE AREA

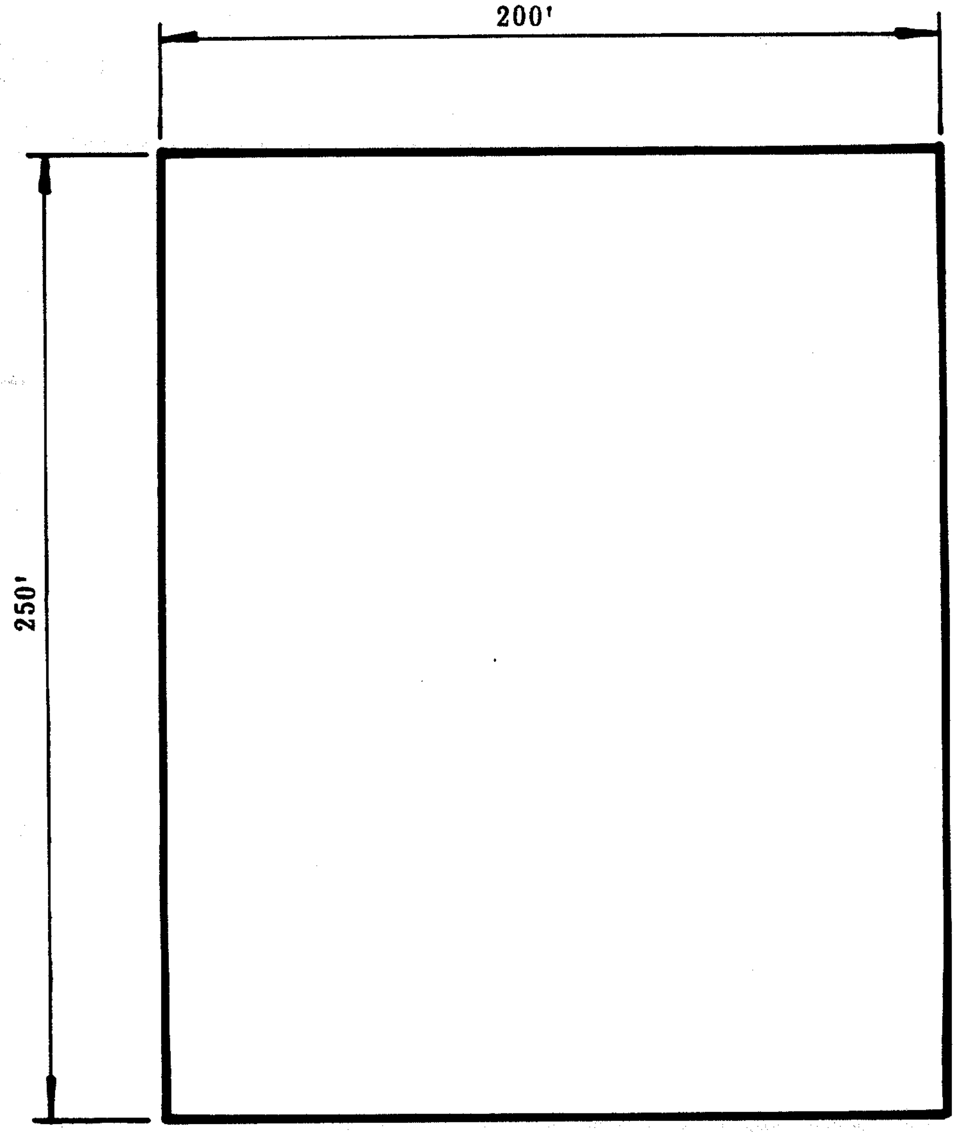


FIGURE 2.6.1 DECK OVERALL DIMENSION

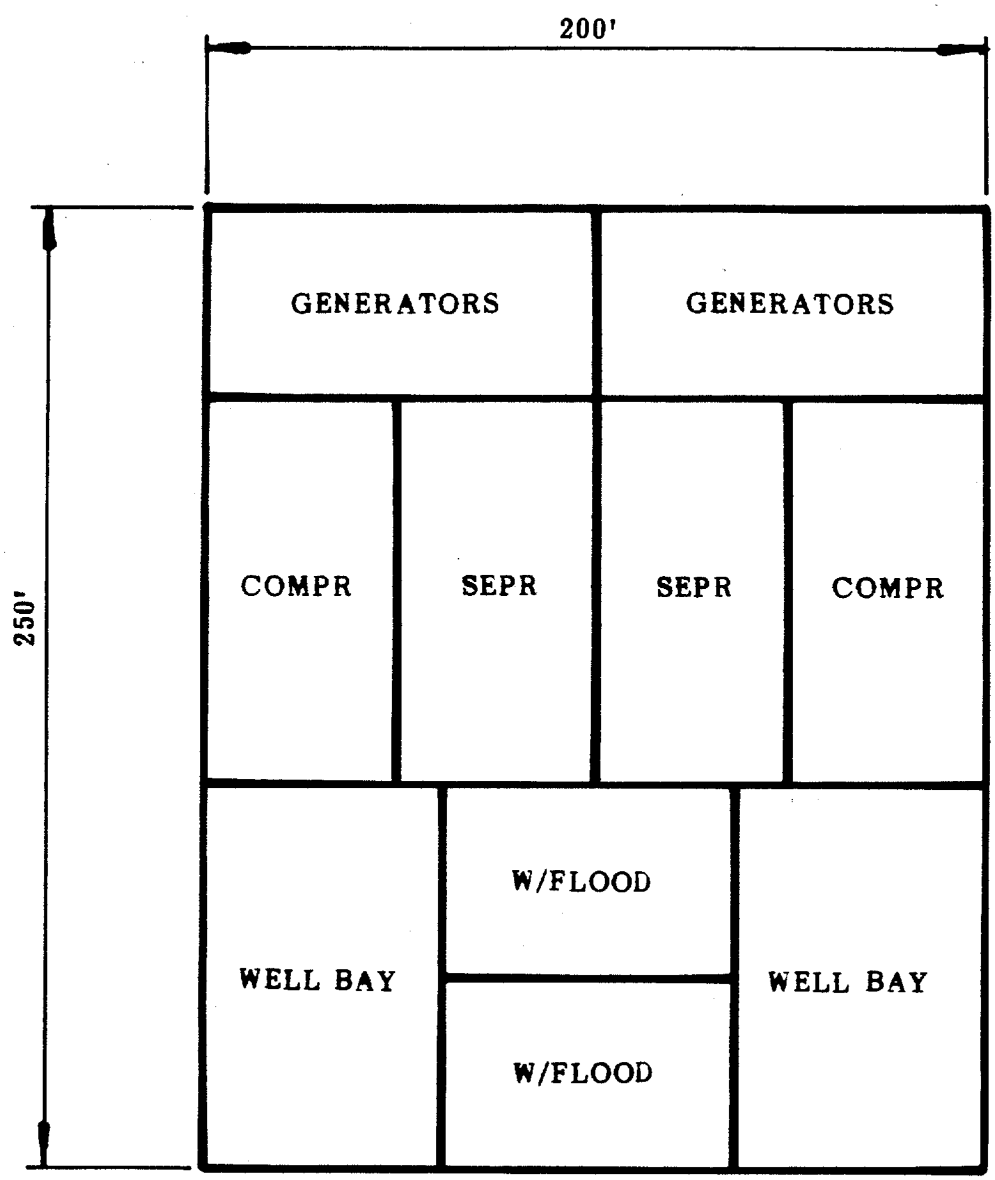


FIGURE 2.6.2 LEVEL 1 DECK LAYOUT

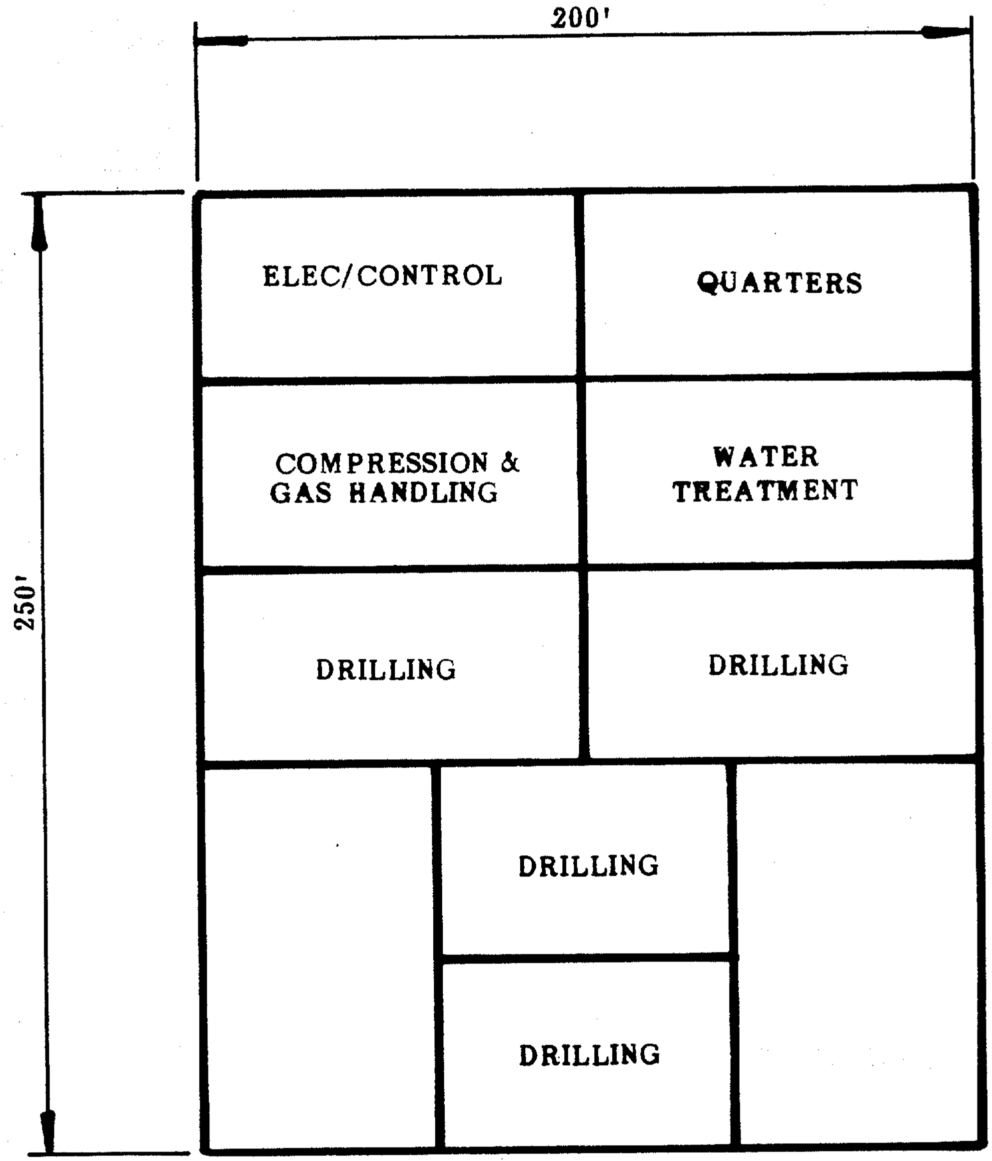


FIGURE 2.6.3 LEVEL 2 DECK LAYOUT

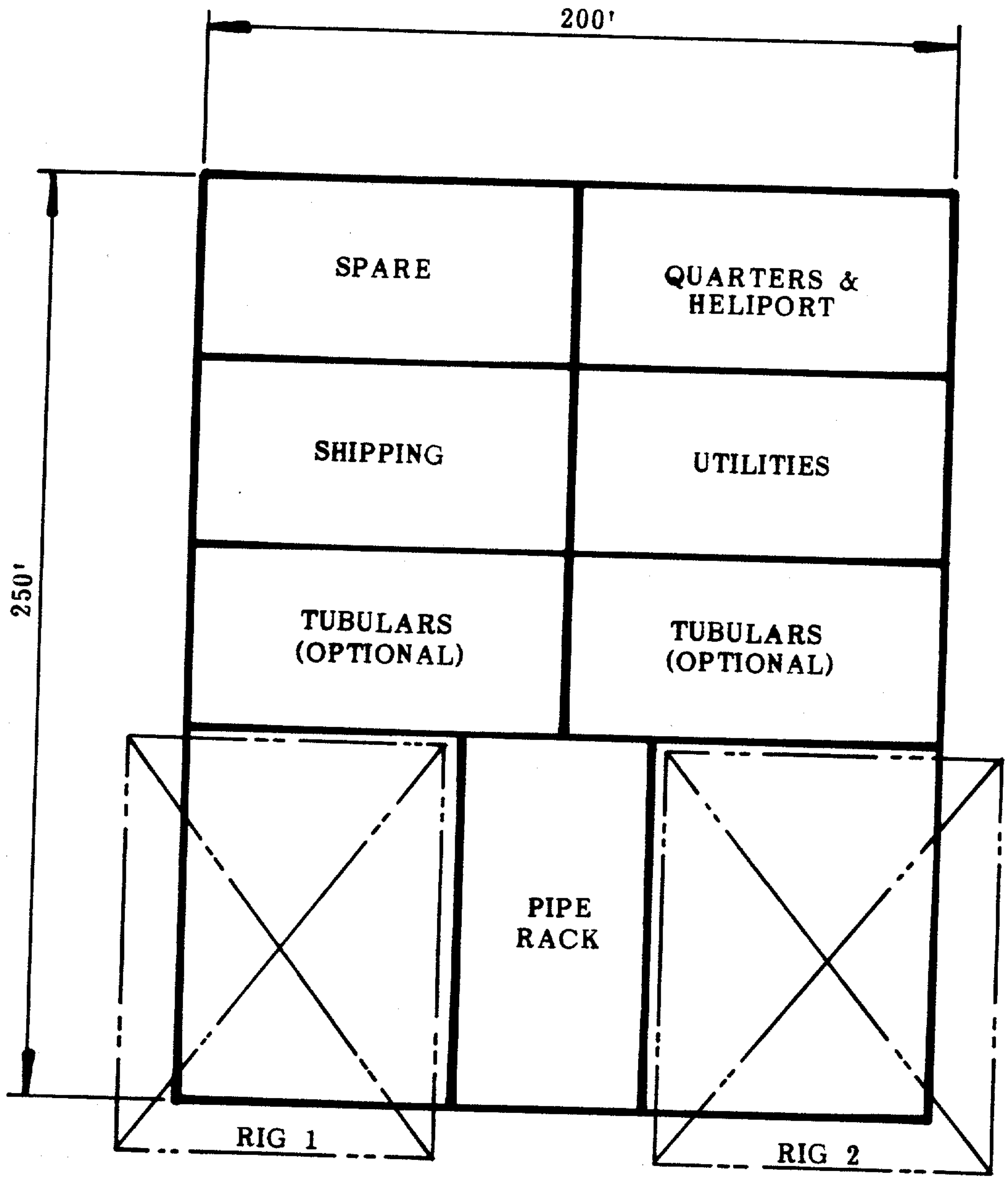


FIGURE 2.6.4 LEVEL 3 DECK LAYOUT

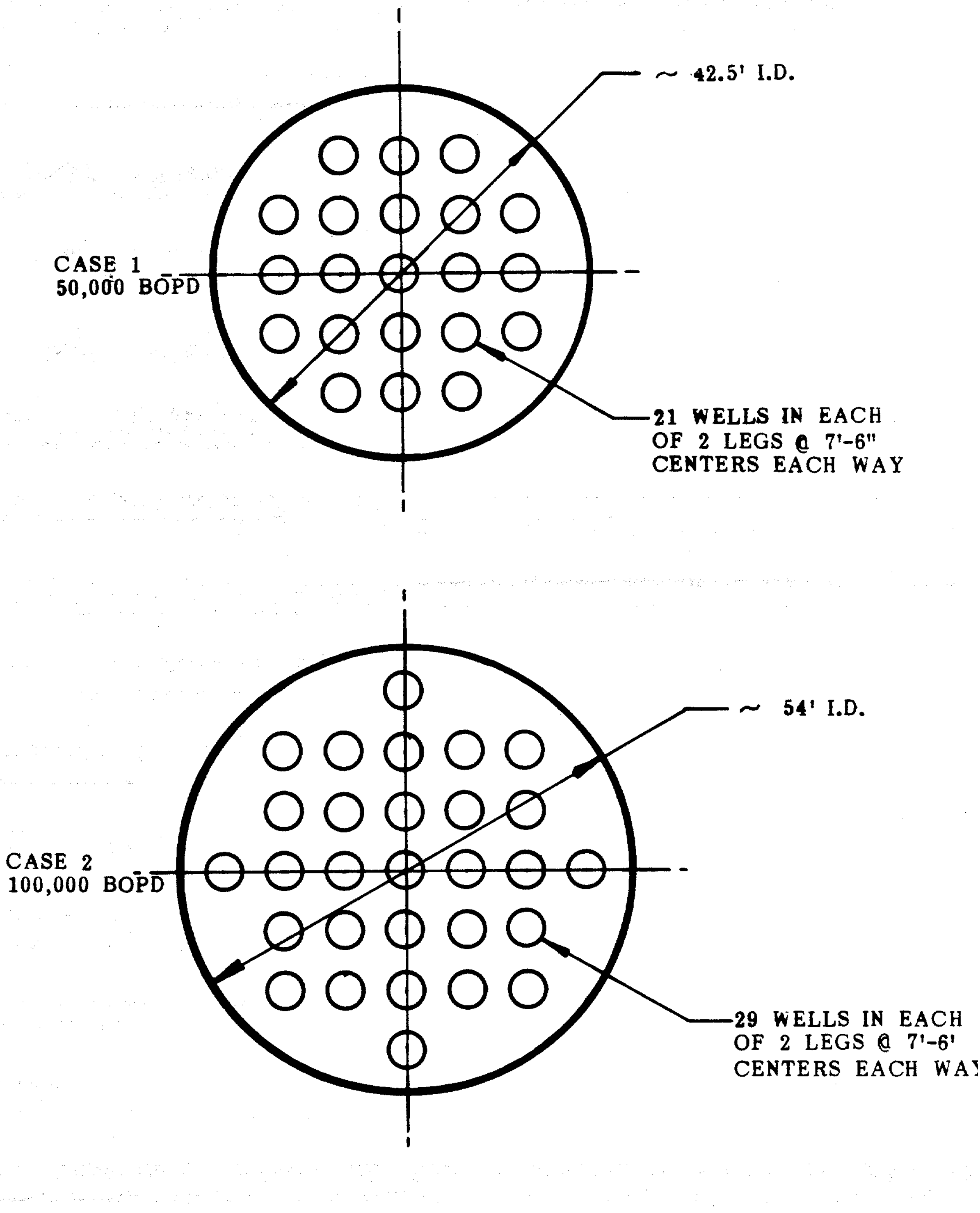


FIGURE 2.6.5 LEG/WELL/PILE ARRANGEMENTS

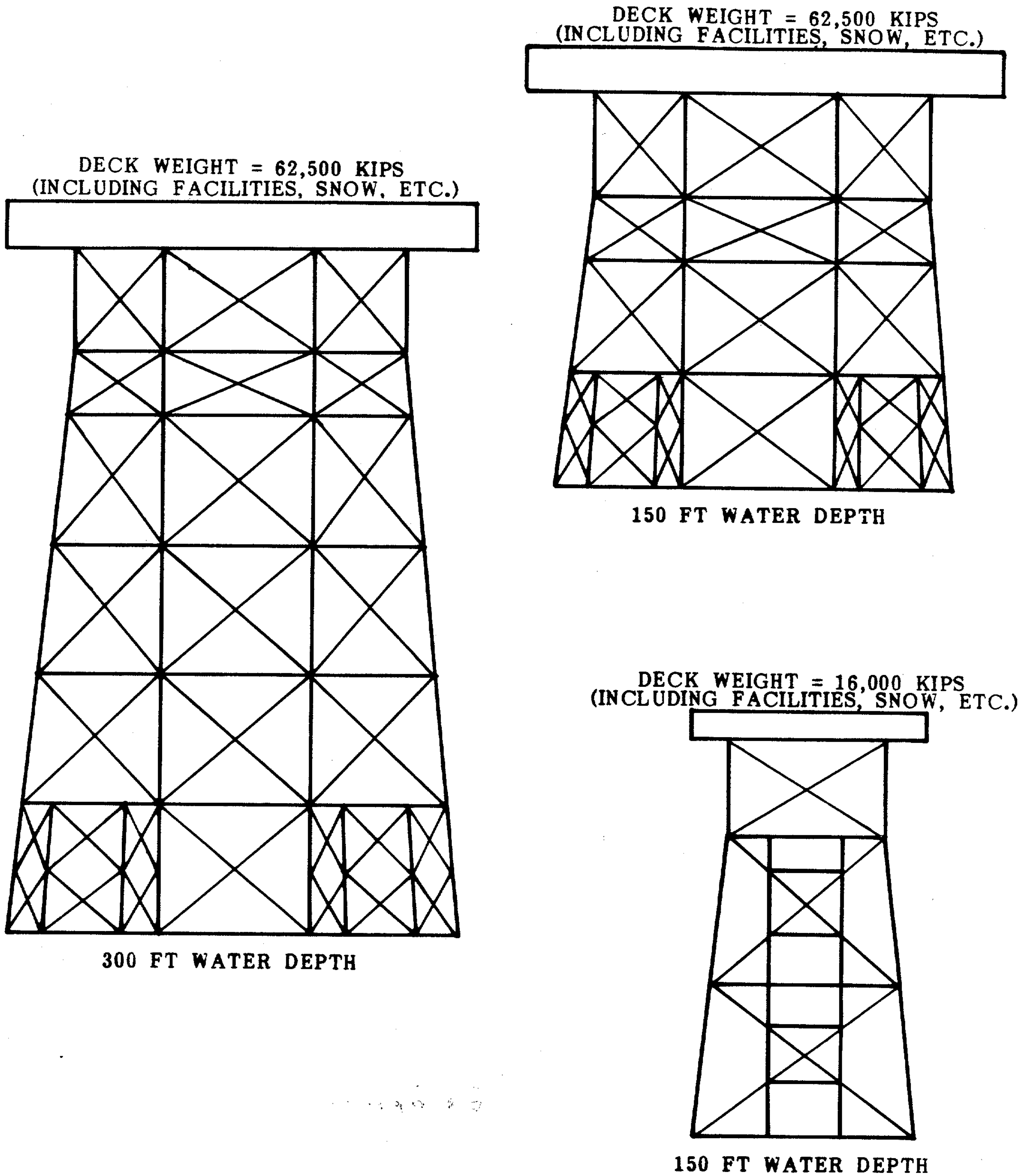


FIGURE 3.1.1 JACKET CONCEPTS

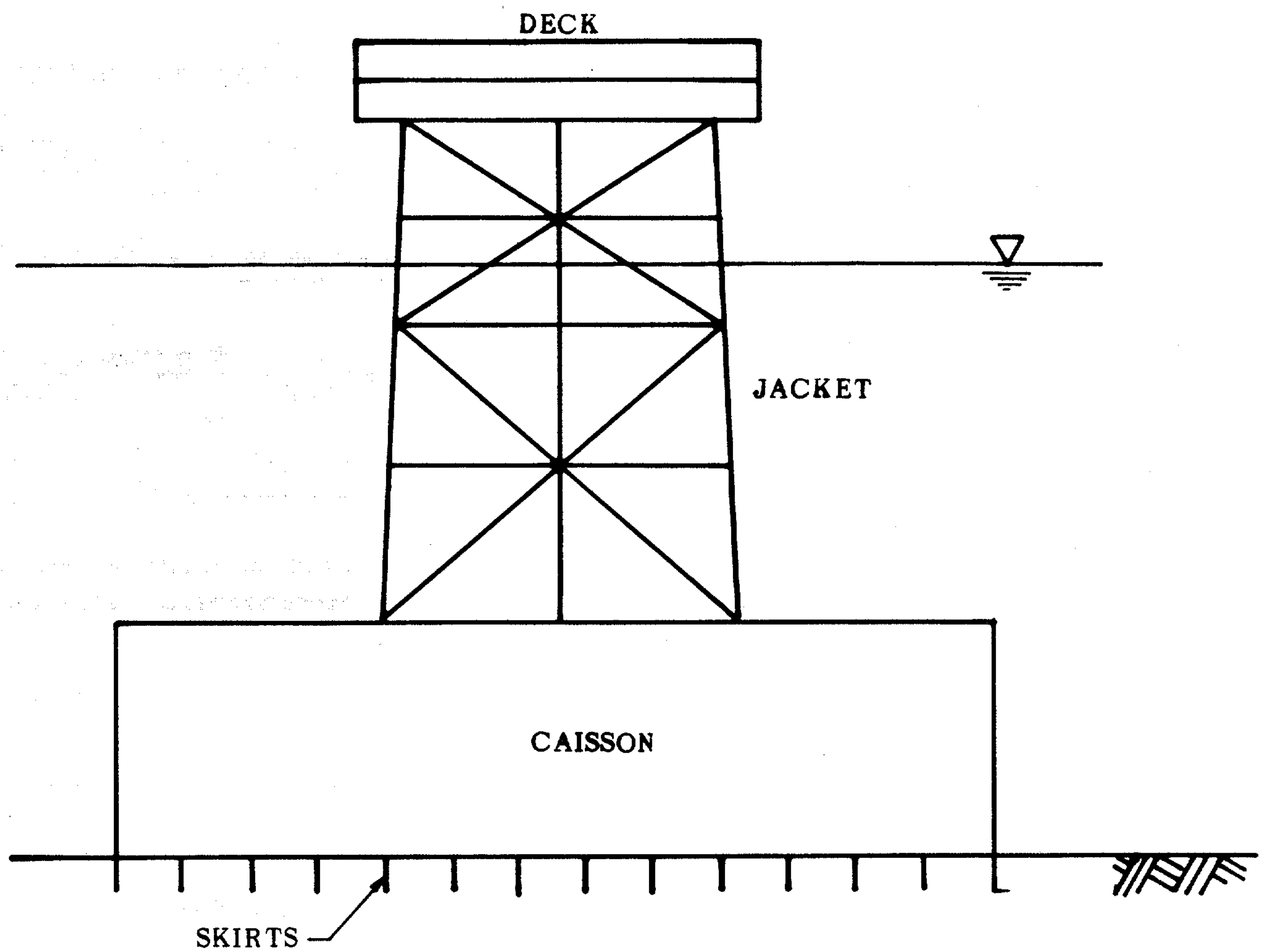
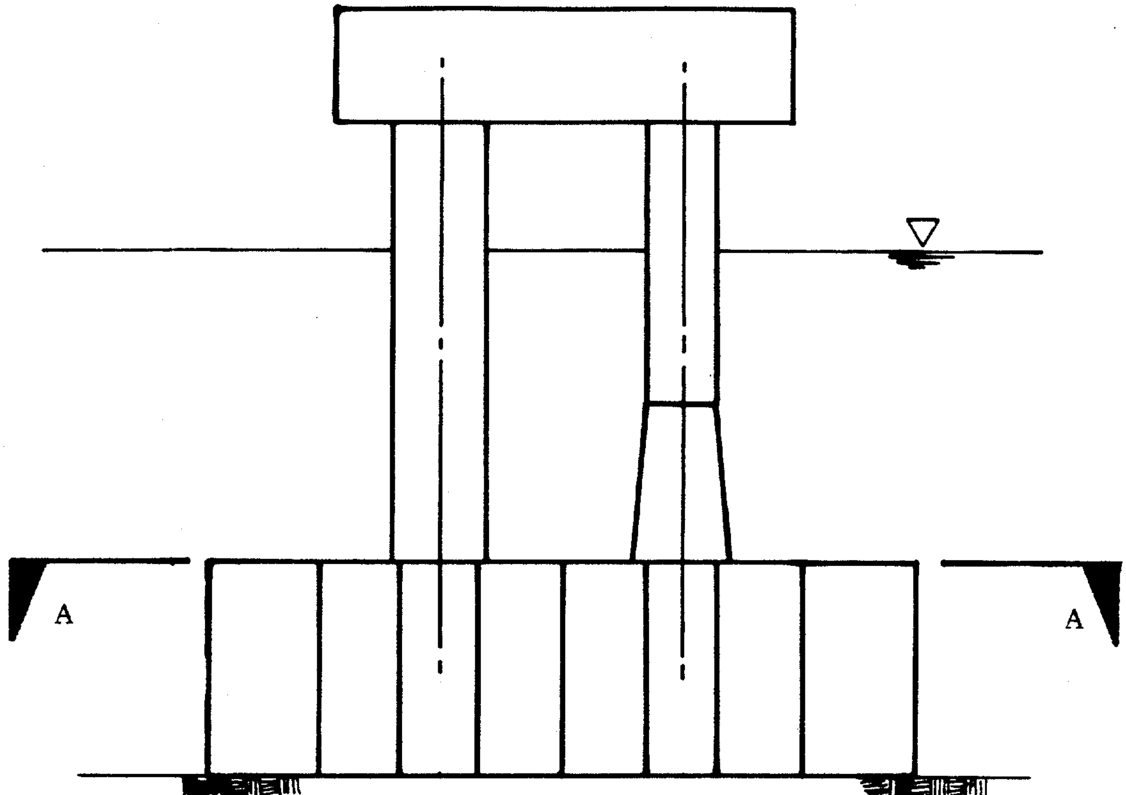
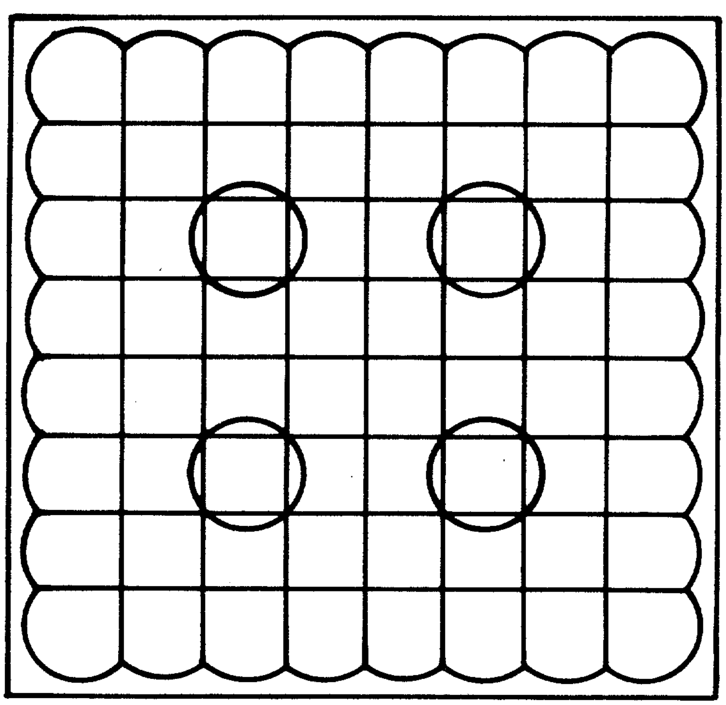


FIGURE 3.2.1 HYBRID STRUCTURE - CONCEPT 2



ELEVATION



SECTION "A-A"

FIGURE 3.3.1 CONCRETE GRAVITY STRUCTURE

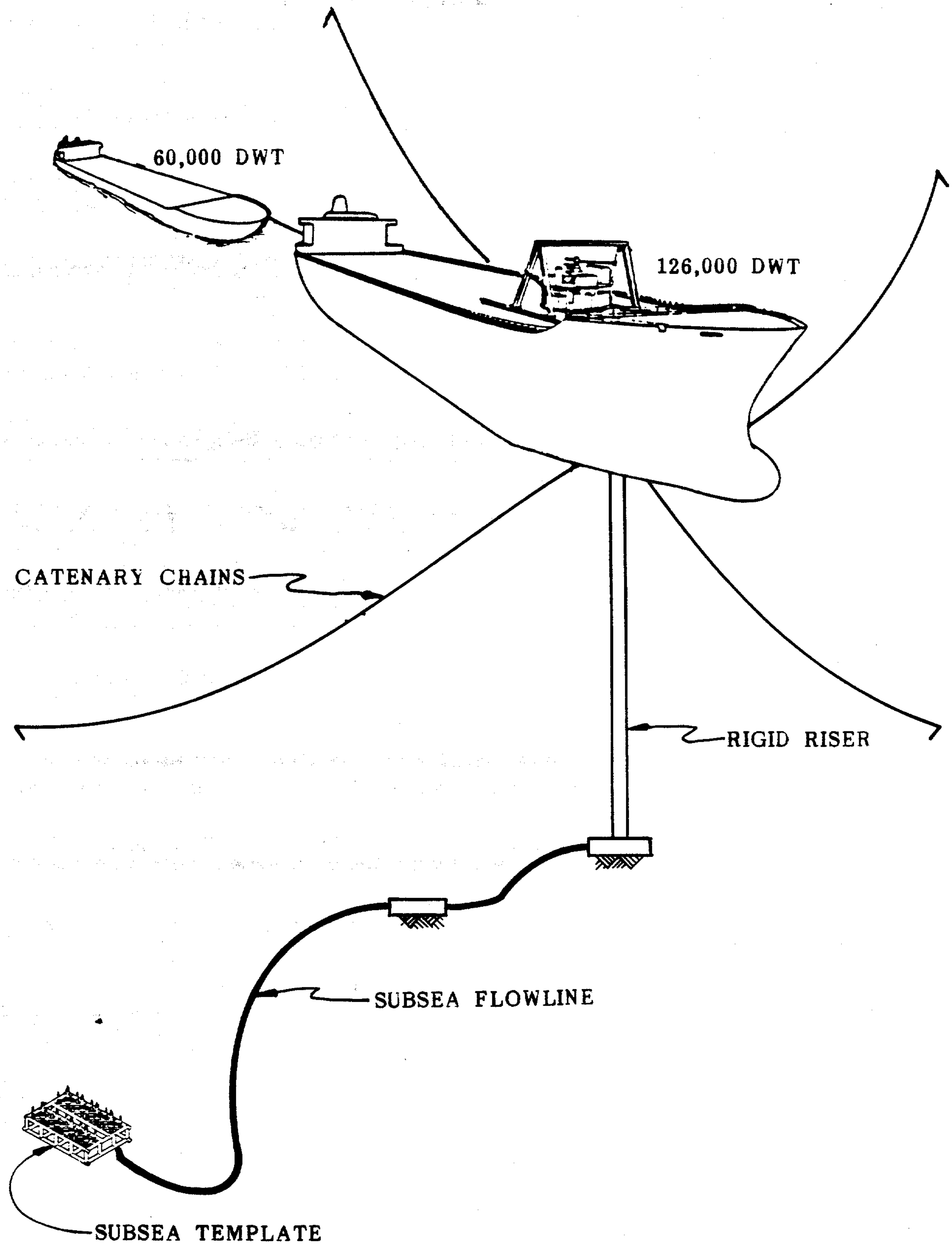


FIGURE 3.4.1 TURRET MOORED RIGID RISER TANKER (TMRR)

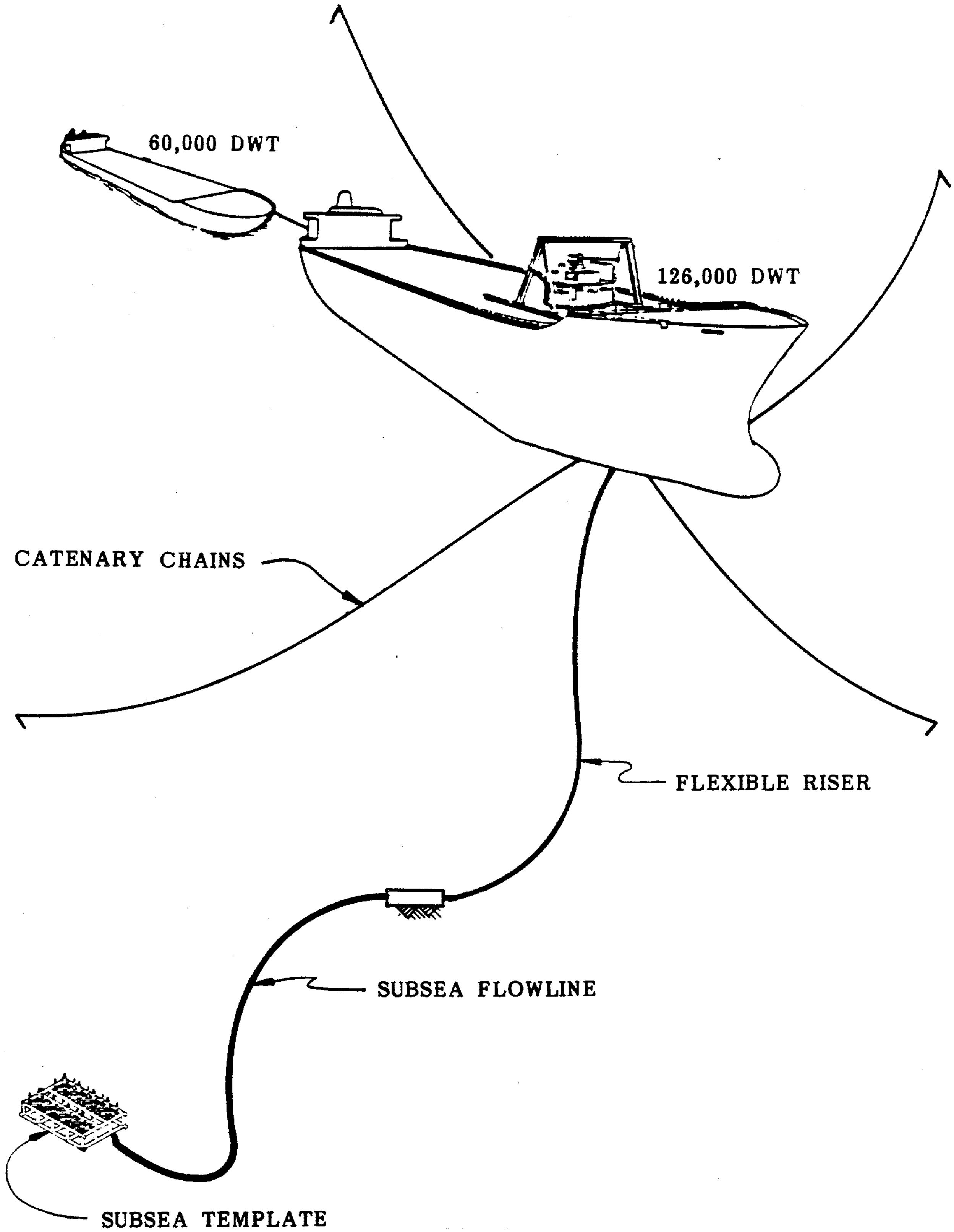


FIGURE 3.4.2 TURRET MOORED FLEXIBLE RISER TANKER (TMFR)

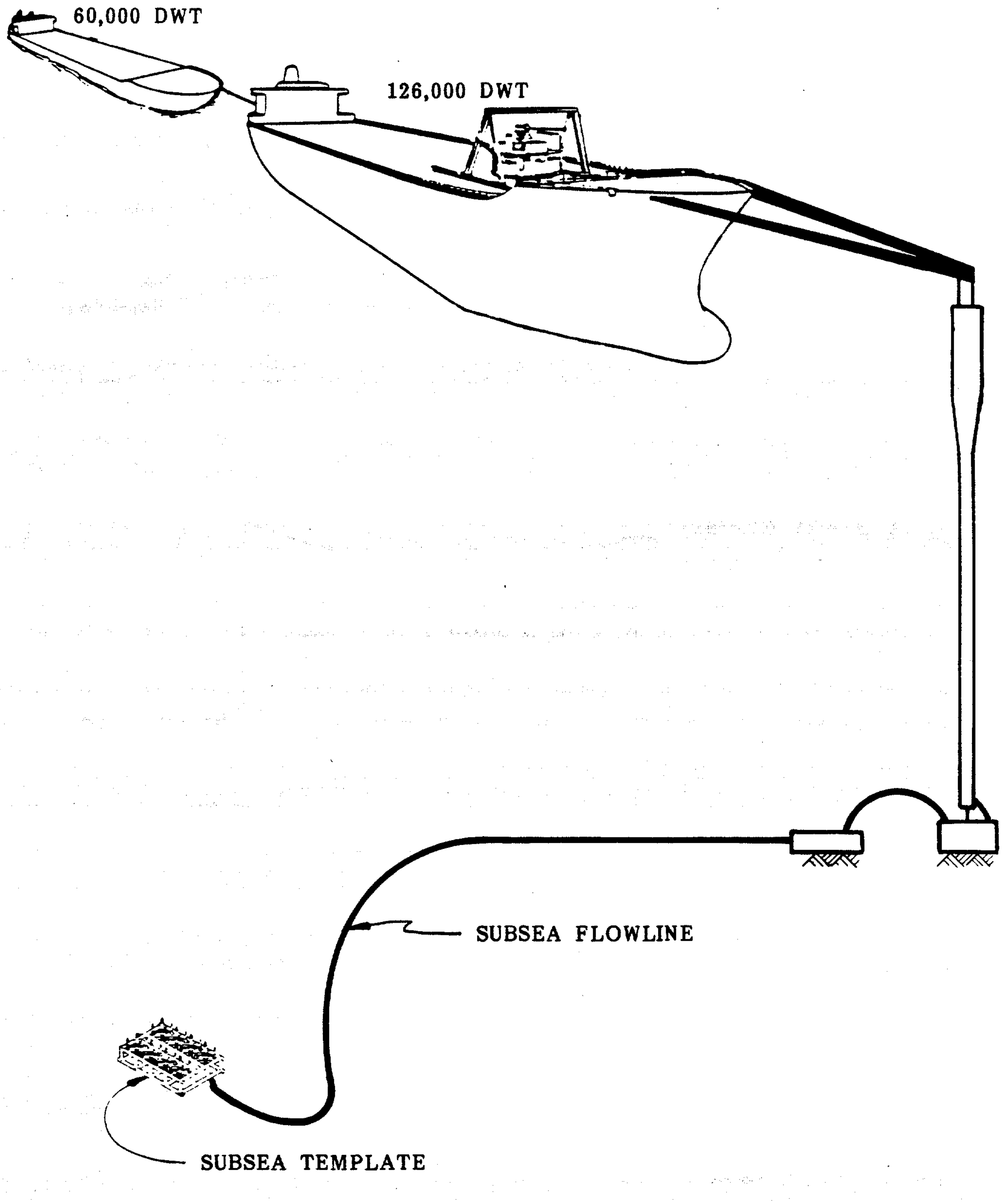


FIGURE 3.4.3 SINGLE ANCHOR LEG MOORING SYSTEM (SALM)

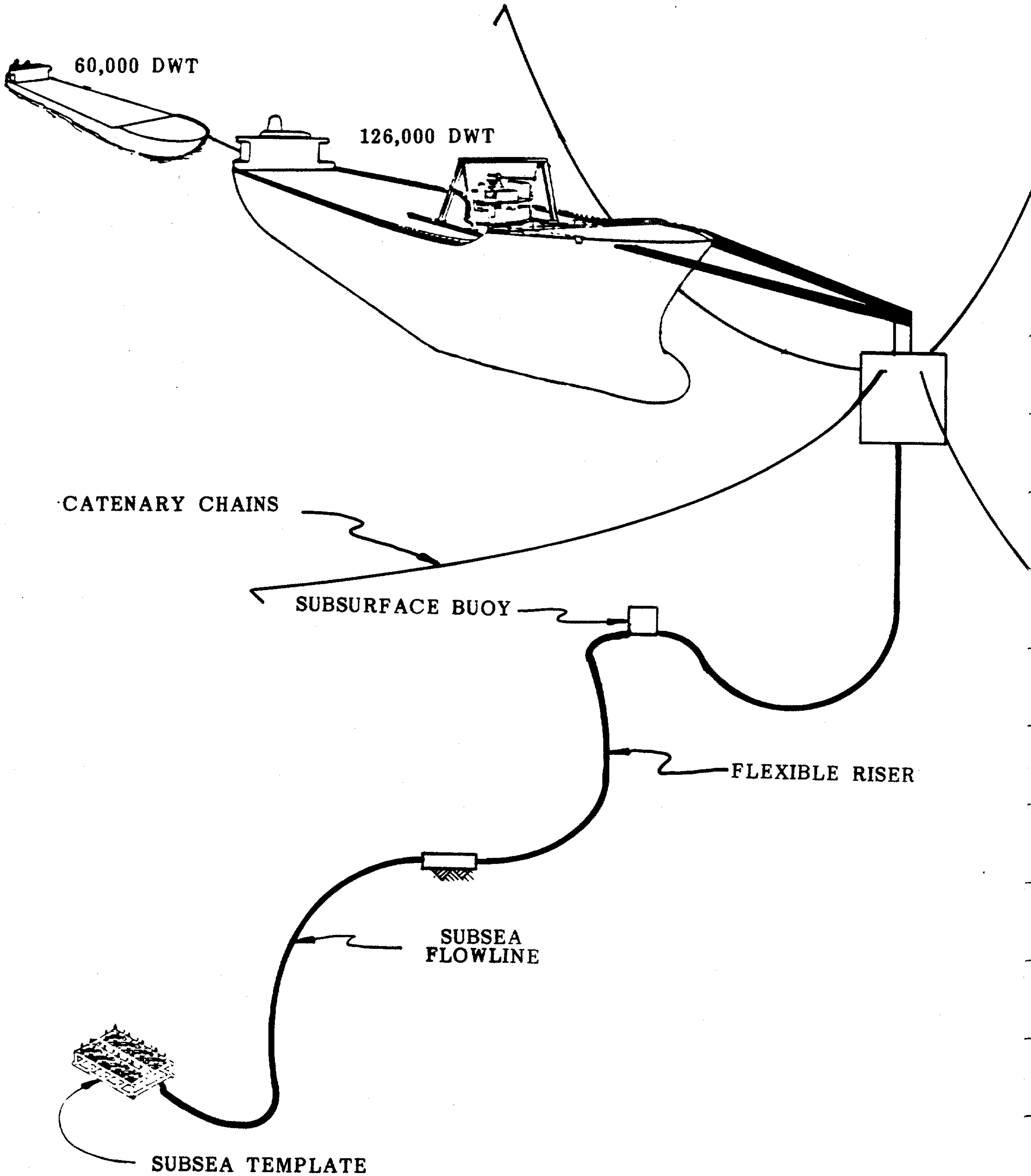


FIGURE 3.4.4 CATENARY ANCHOR LEG MOORING SYSTEM (CALM)

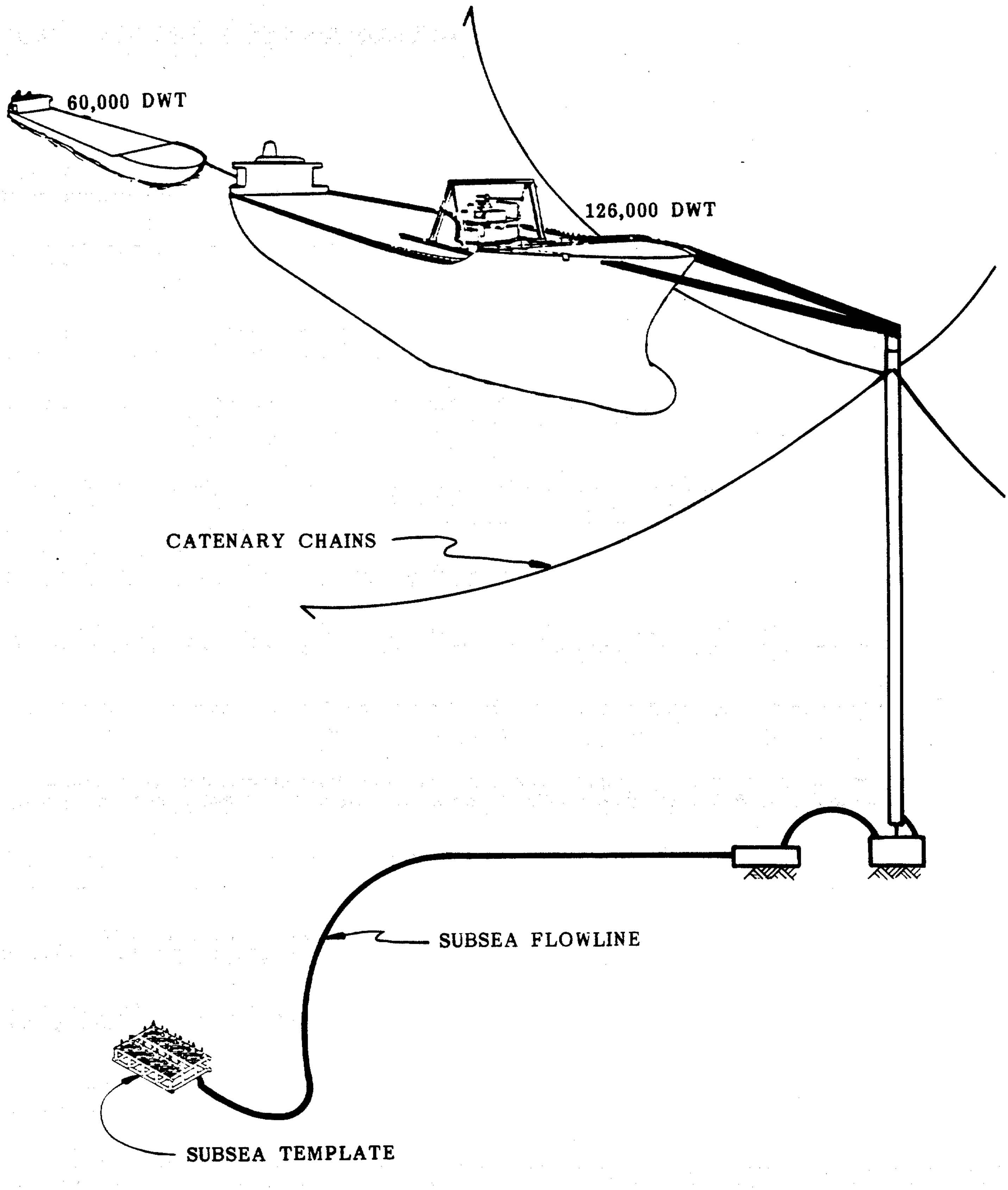


FIGURE 3.4.5 CATENARY ANCHORED TOWER (CAT)

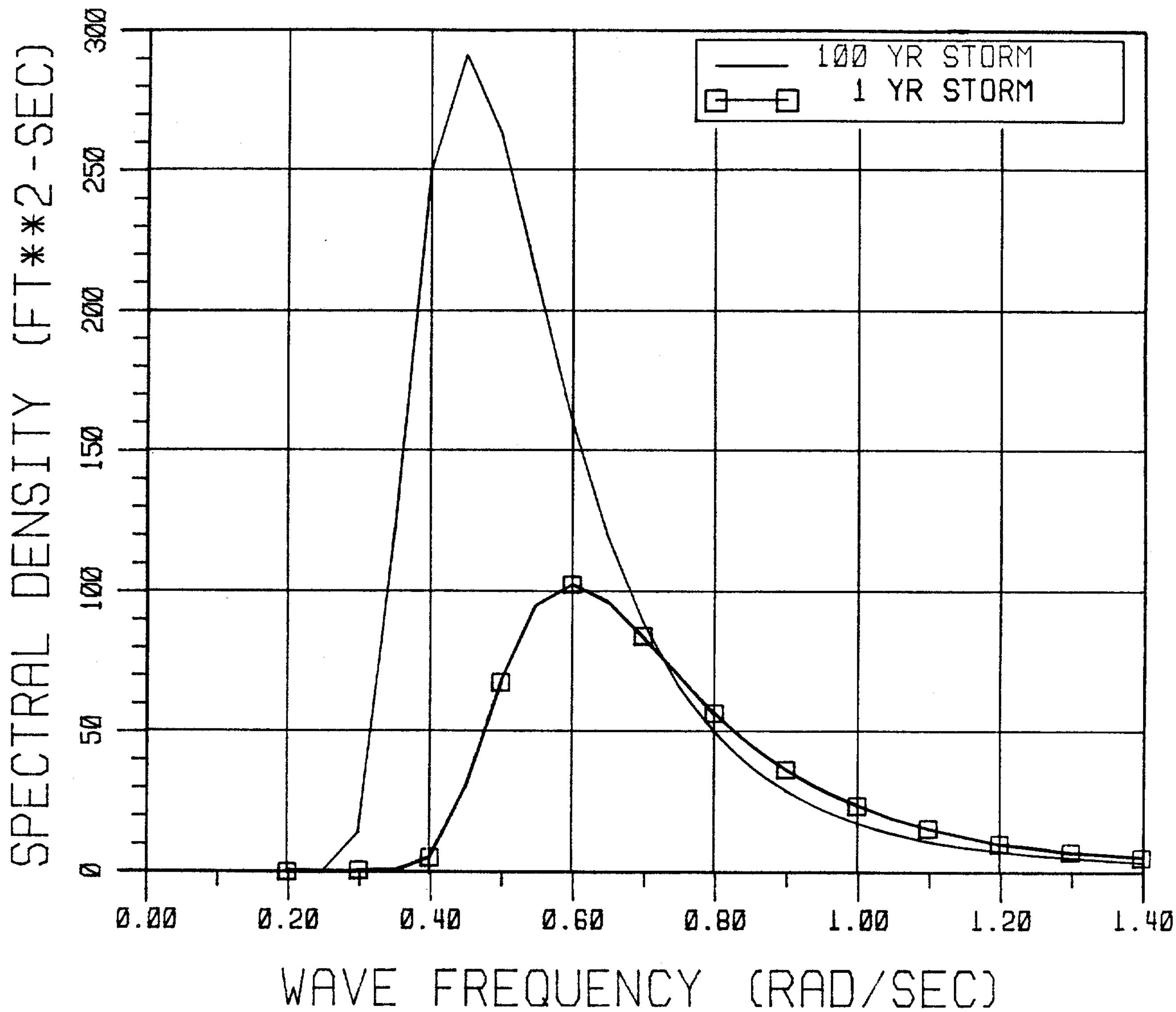


FIGURE 3.4.6 <u>**JONSWAP SPECTRAL VALUES**</u>

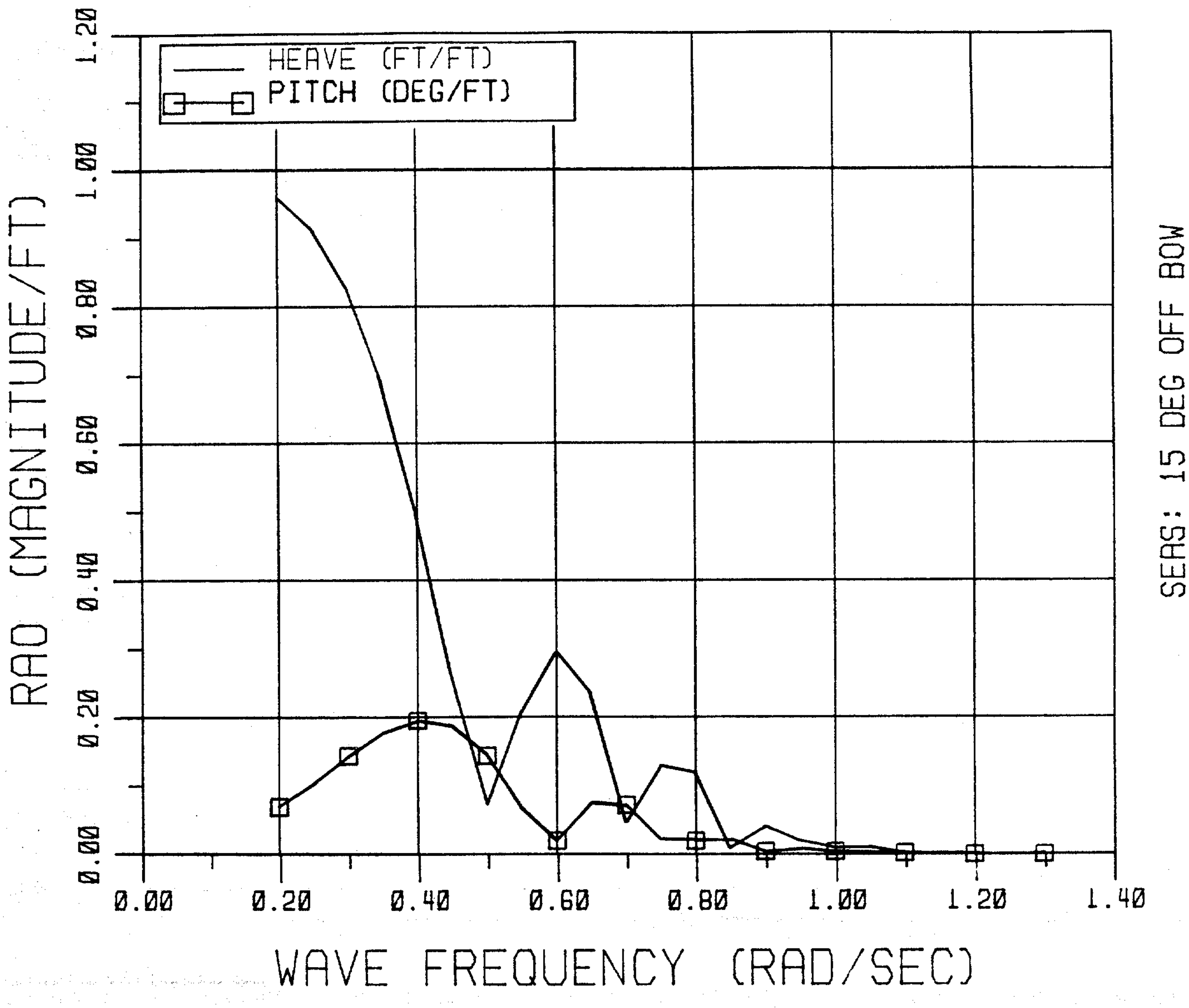


FIGURE 3.4.7 RAO VALUES FOR CAPTIVE TANKER

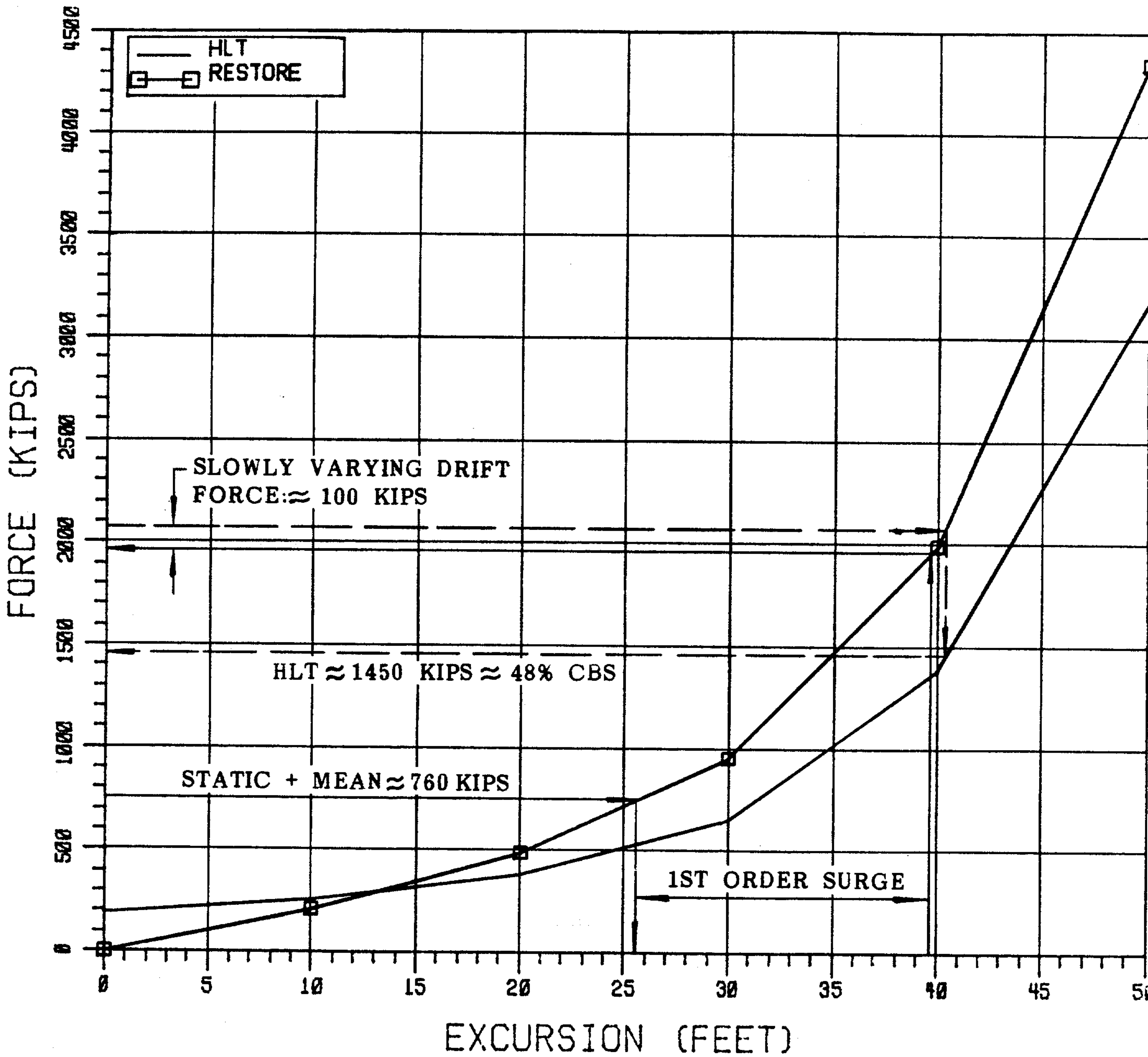


FIGURE 3.4.8 CAPTIVE TANKER MOORING FORCES

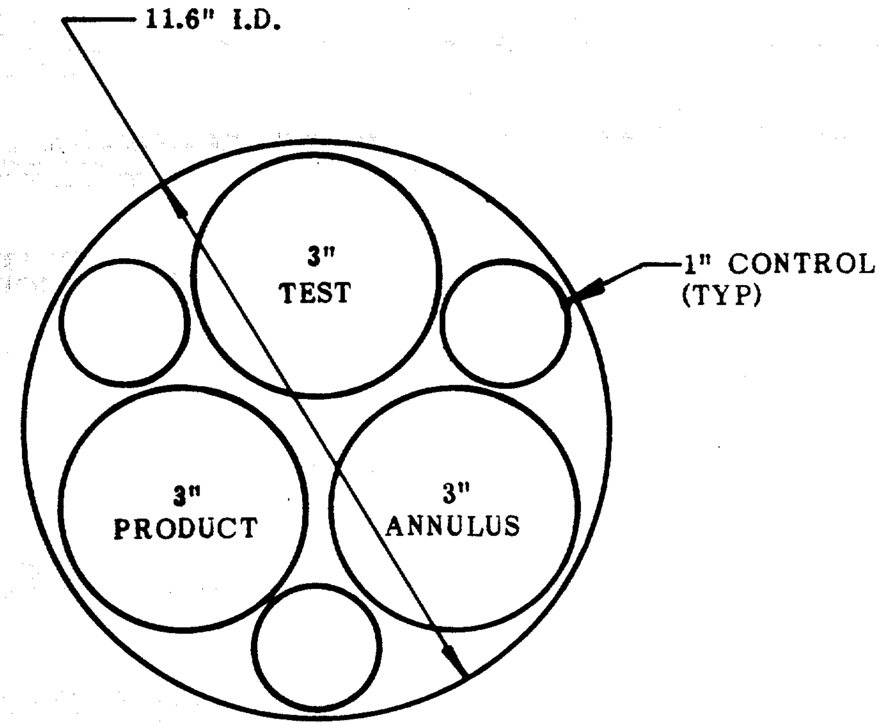


FIGURE 3.4.9 FLEXIBLE RISER SYSTEM

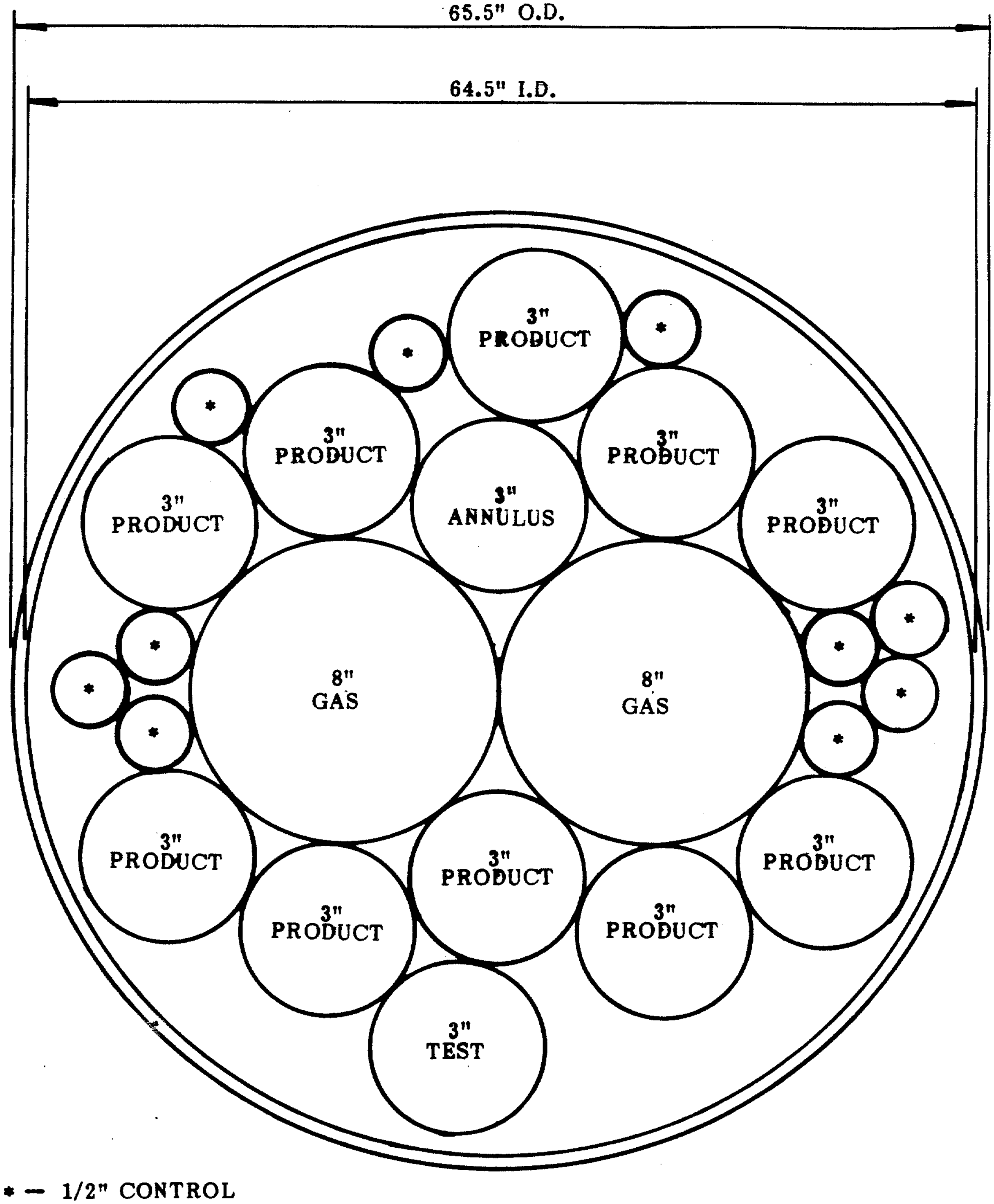


FIGURE 3.4.10 RIGID RISER CONFIGURATION

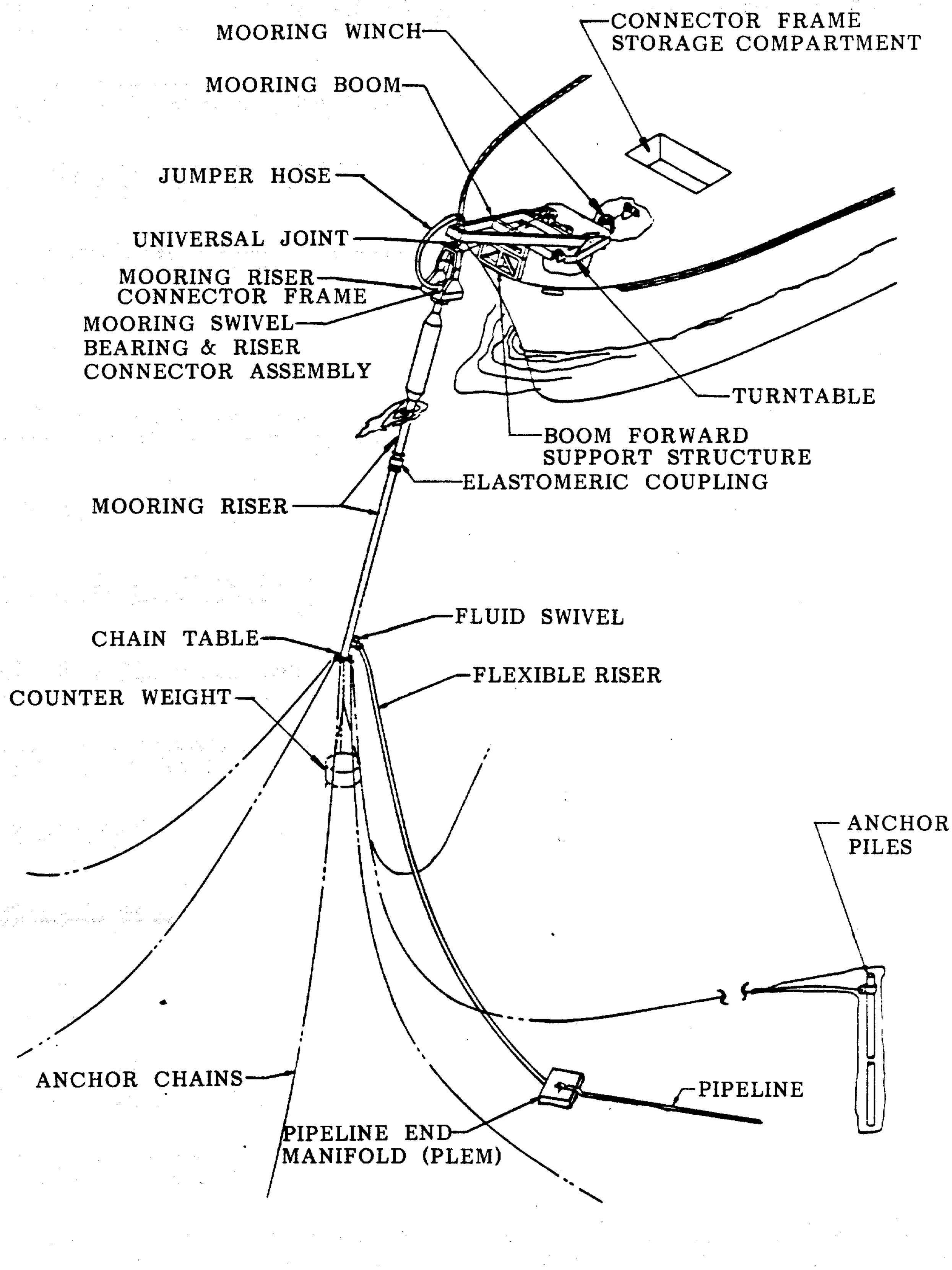


FIGURE 3.5.1 COUNTERWEIGHT ANCHOR MOORING (CAM) SYSTEM FOR REMOTE LOADING

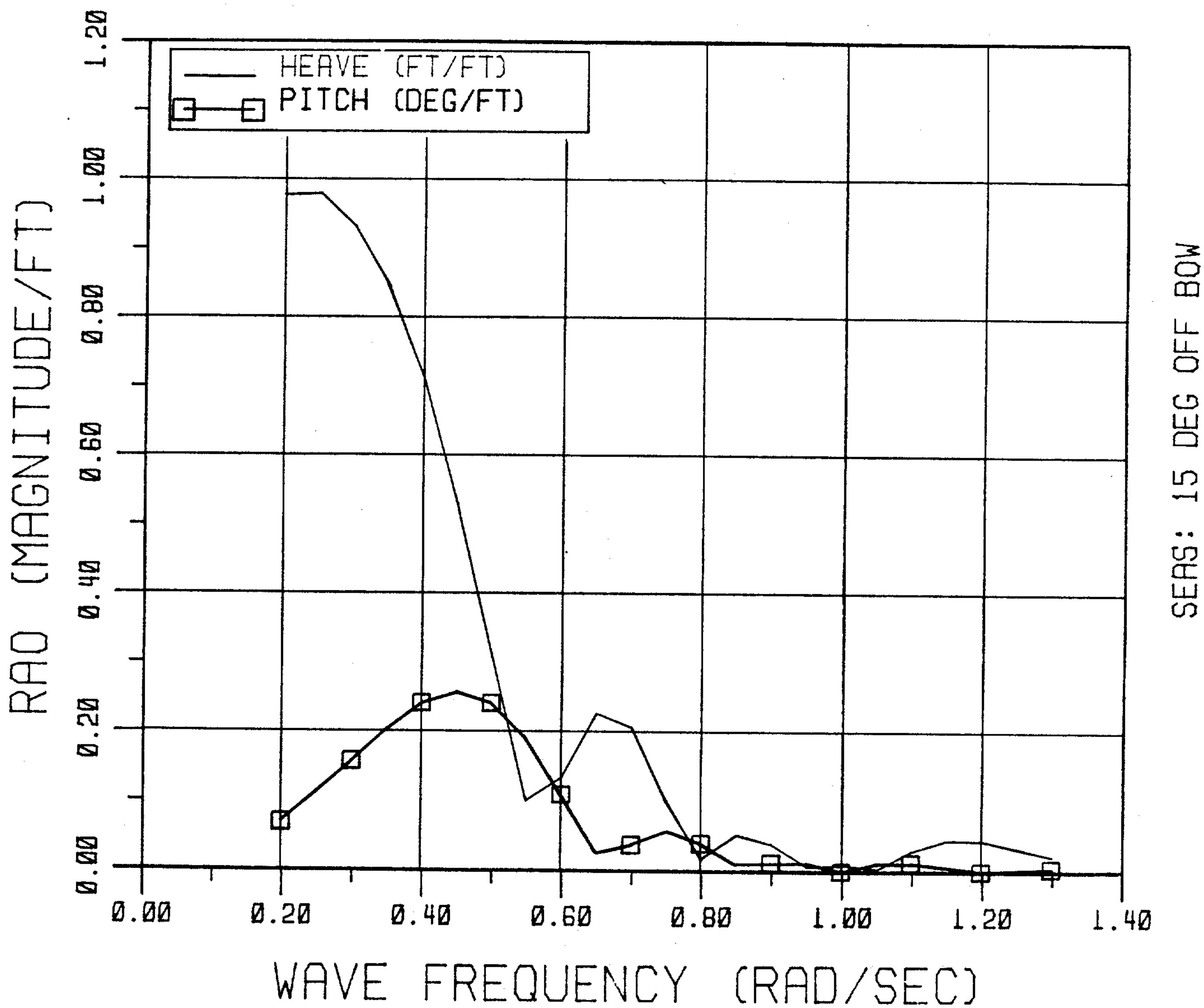


FIGURE 3.5.2 <u>SHUTTLE TANKER RAO'S</u>

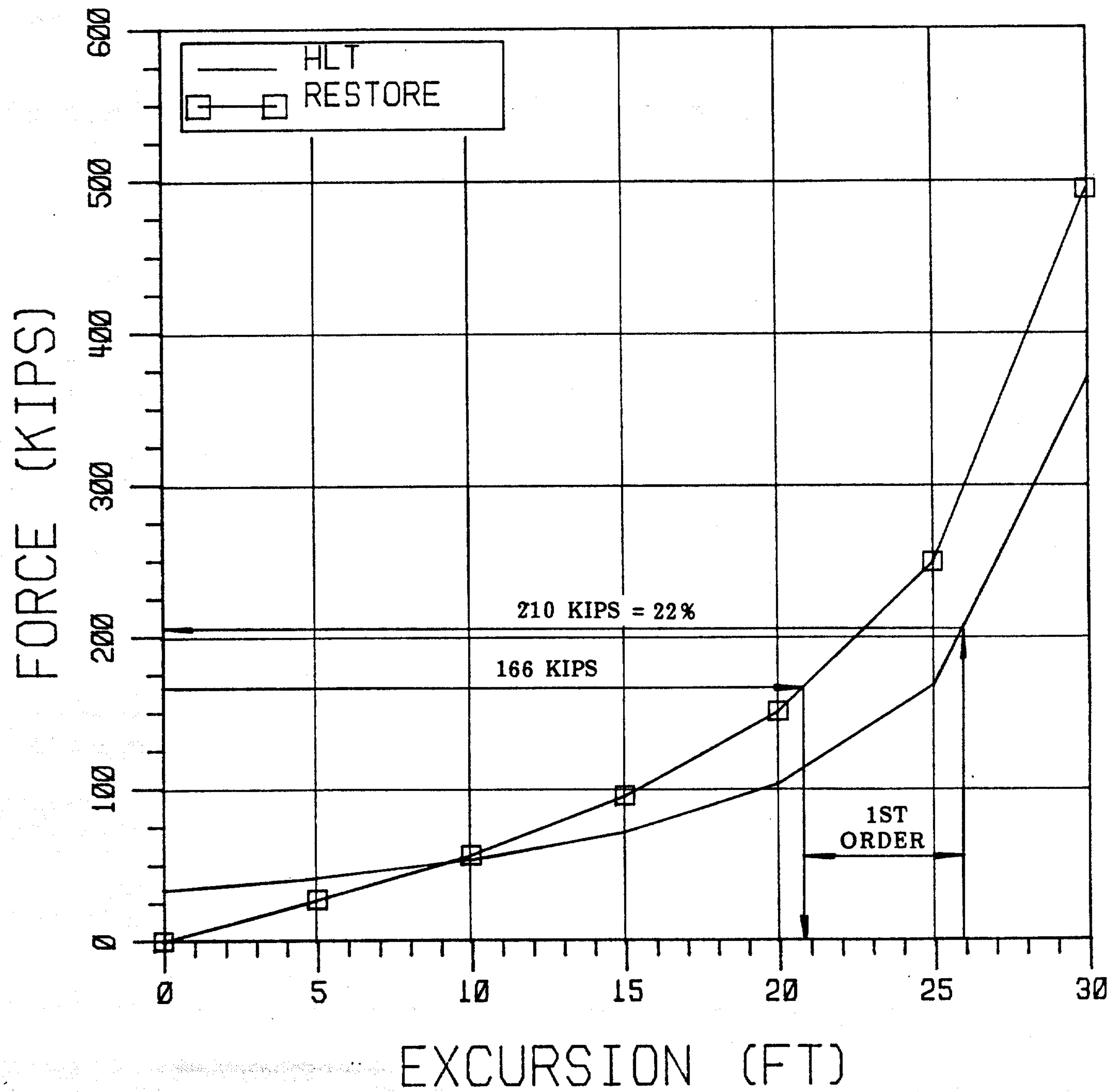


FIGURE 3.5.3 SHUTTLE TANKER MOORING LOADS

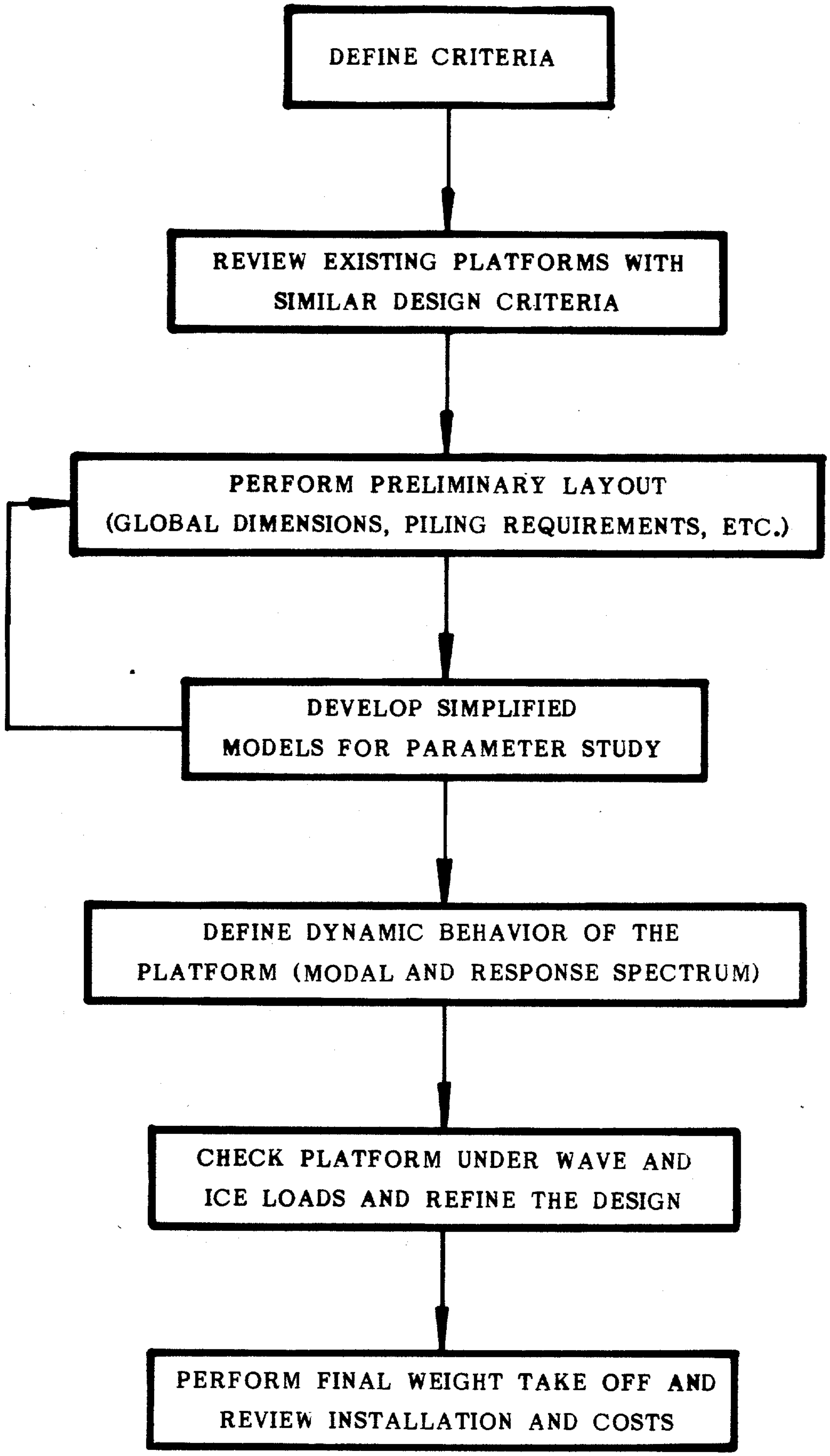


FIGURE 5.0.1 DESIGN SEQUENCE

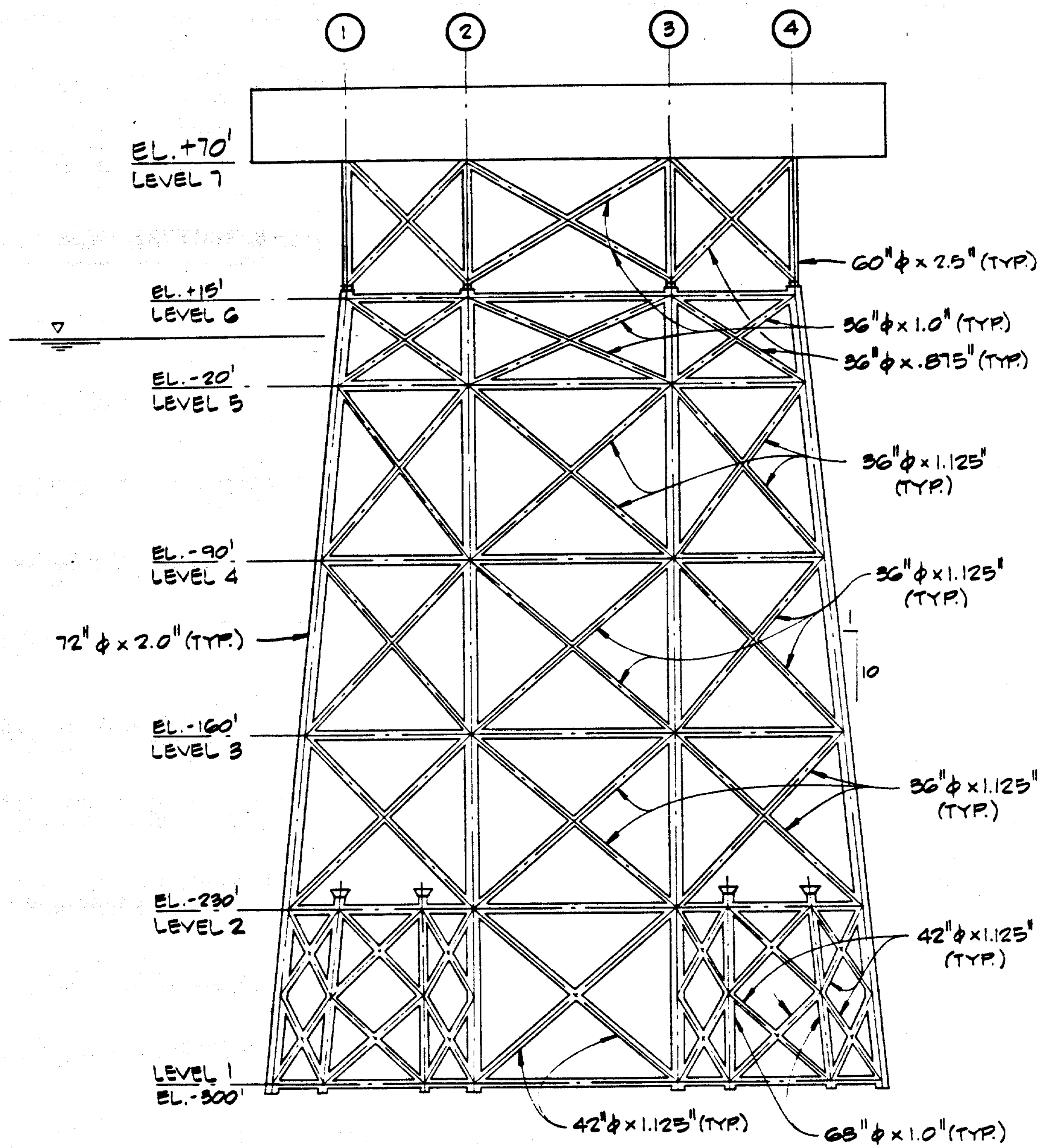


FIGURE 5.1.1 300 FT WATER DEPTH-BASE CASE

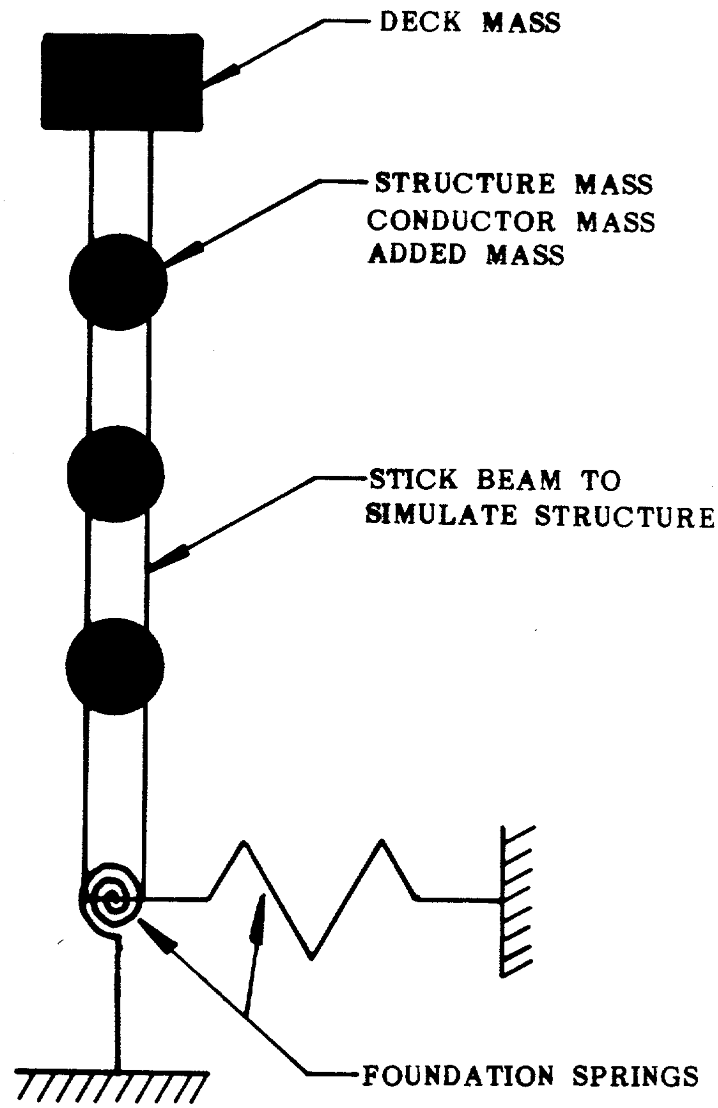


FIGURE 5.1.2 SIMPLIFIED "STICK" MODEL FOR PARAMETRIC STUDIES

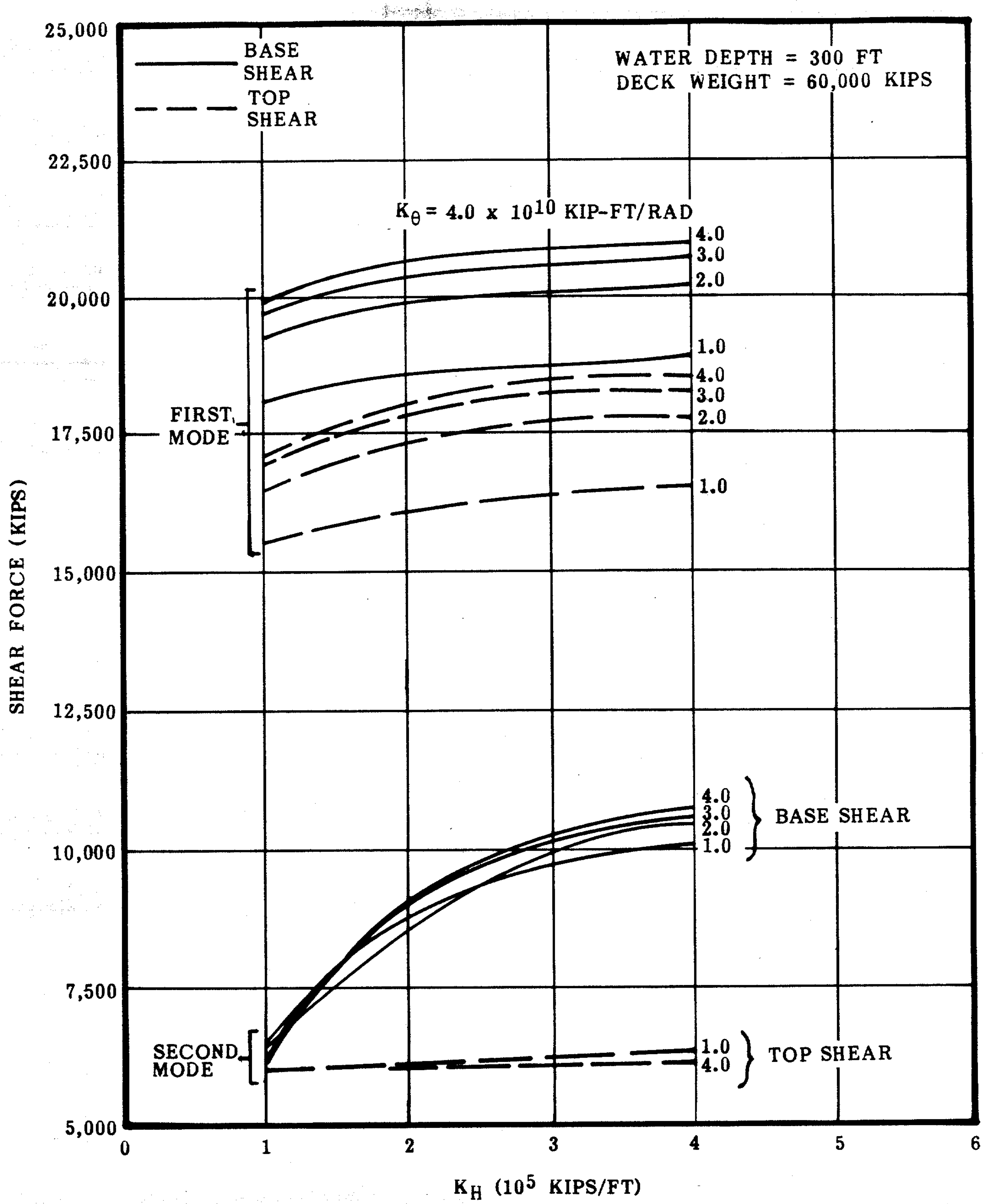


FIGURE 5.1.3 BASE SHEAR AS A FUNCTION OF SOIL SPRING STIFFNESS, 300 FT DEPTH

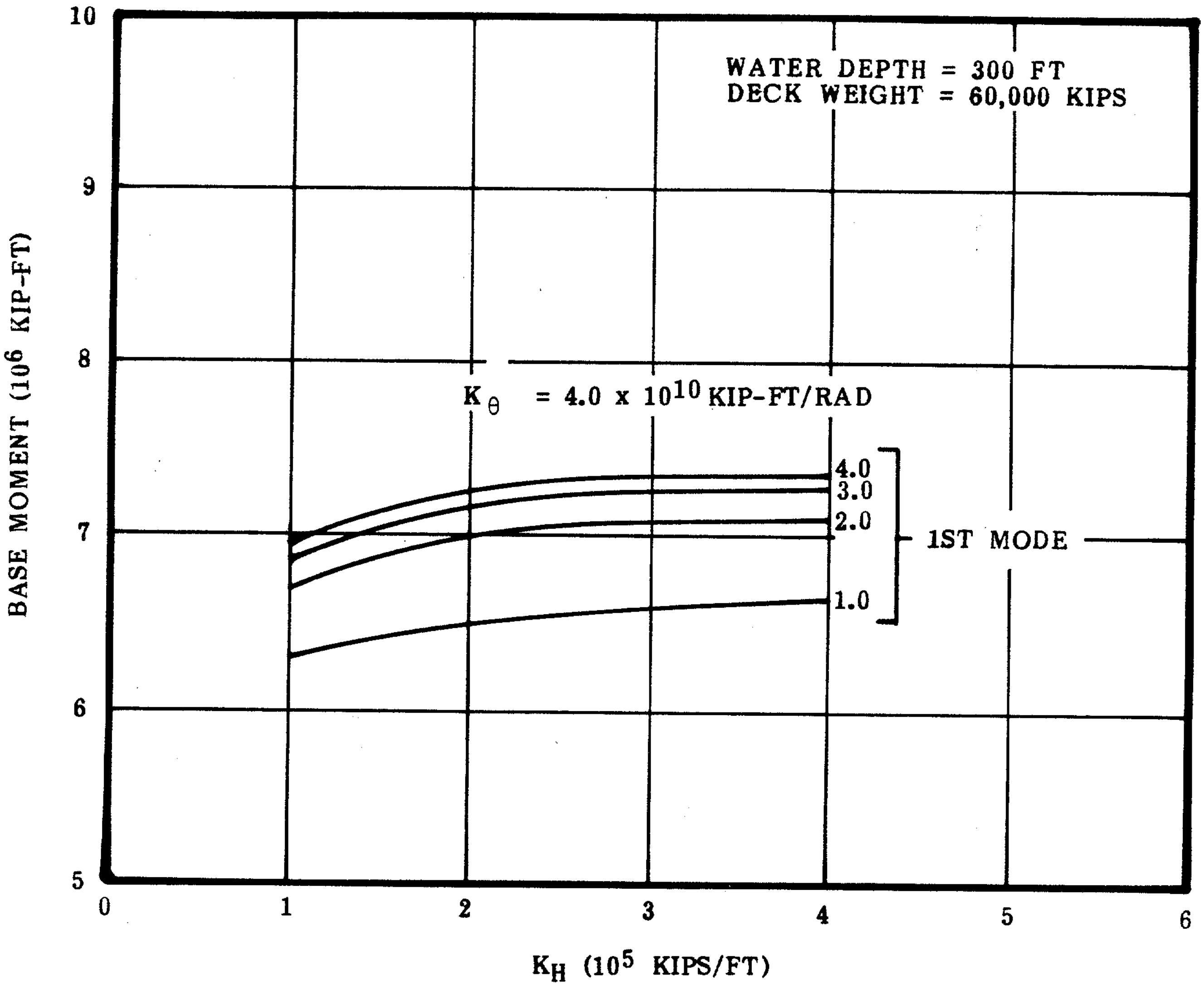


FIGURE 5.1.4 BASE OVERTURNING MOMENT AS A FUNCTION OF SOIL SPRING STIFFNESS, 300 FT DEPTH

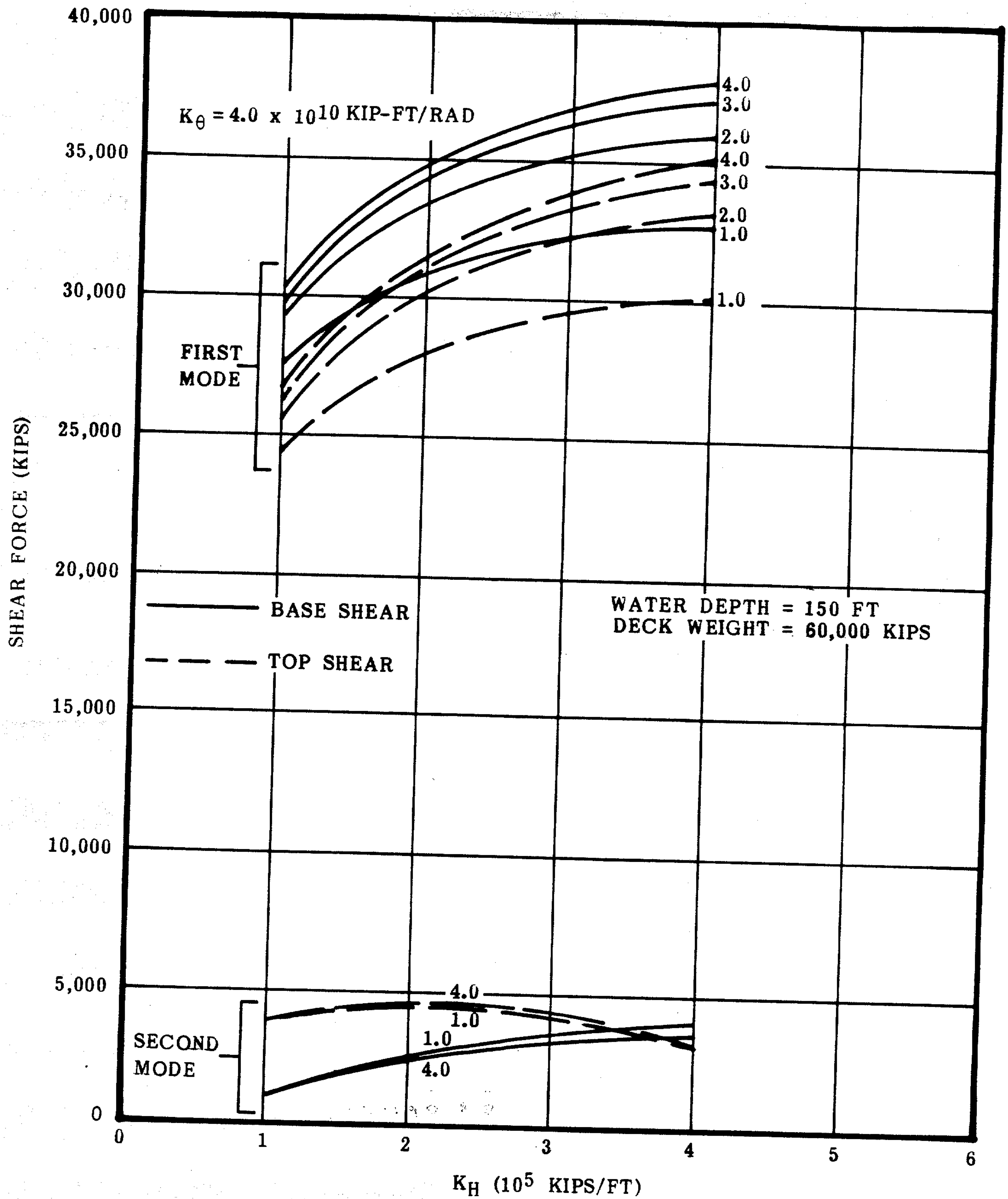


FIGURE 5.1.5 BASE SHEAR AND DECK SHEAR AS A FUNCTION OF SOIL SPRING STIFFNESS, 150 FT DEPTH

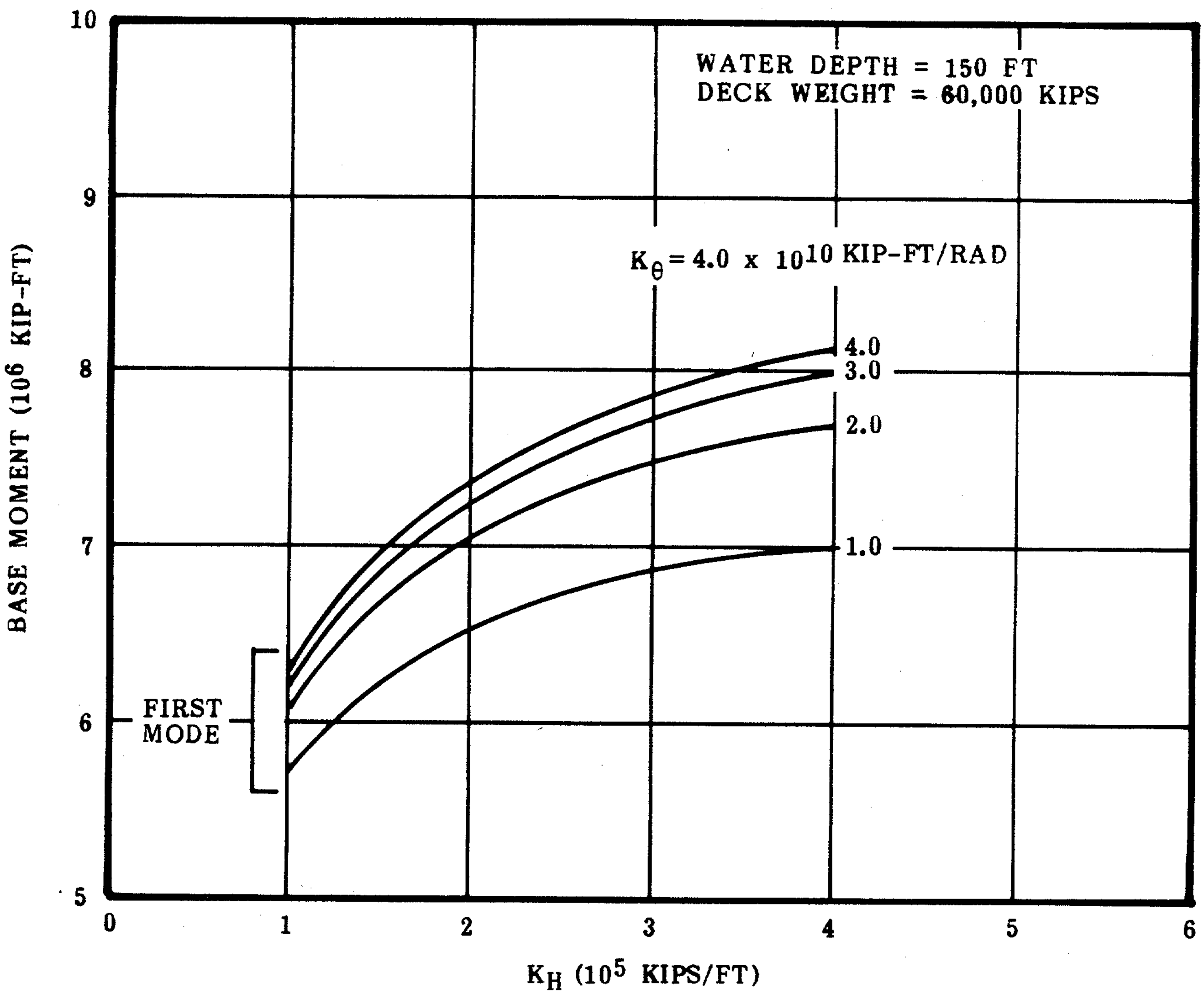


FIGURE 5.1.6 BASE OVERTURNING MOMENT AS A FUNCTION OF SOIL SPRING STIFFNESS, 150 FT DEPTH

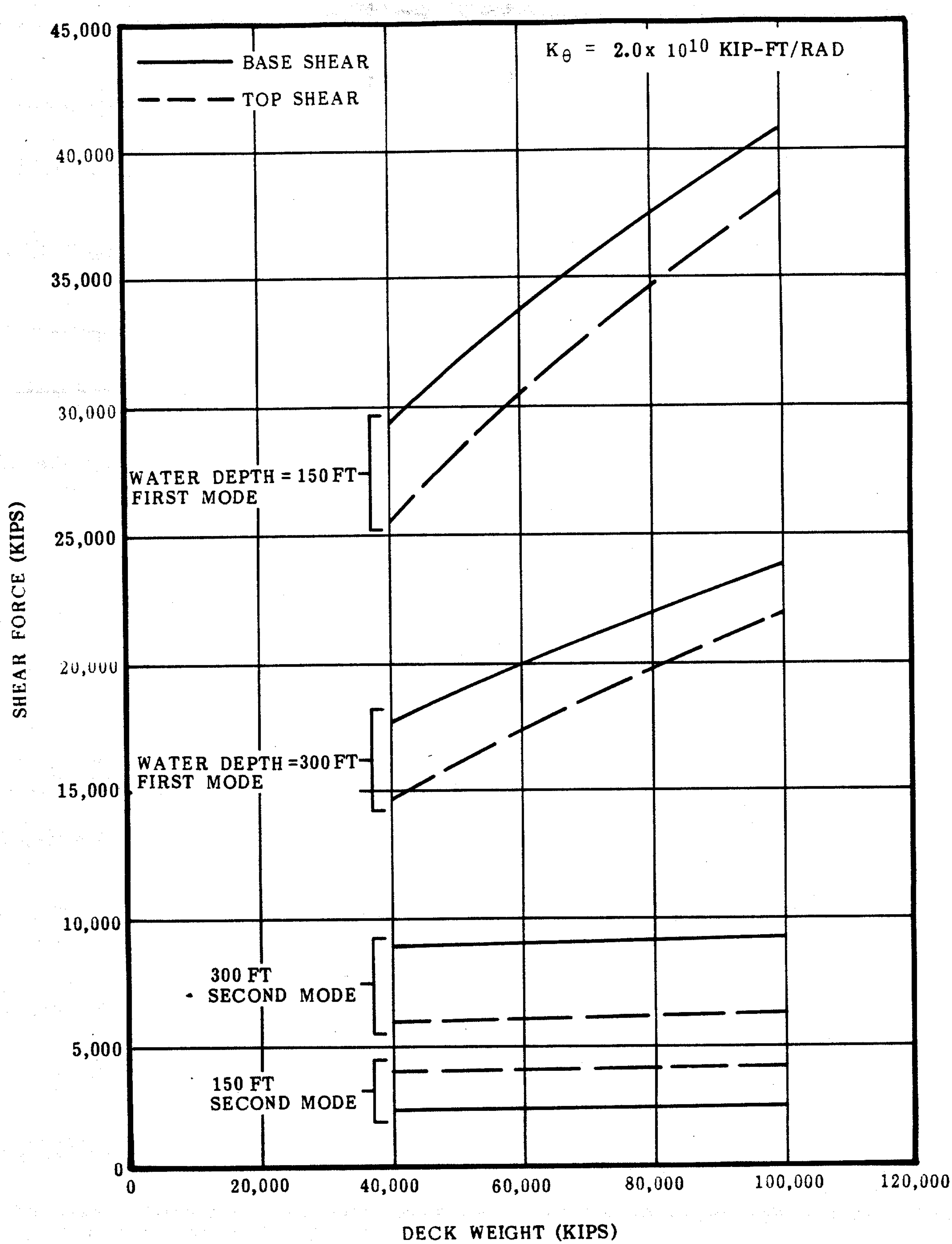


FIGURE 5.1.7 BASE AND DECK SHEAR AS A FUNCTION OF DECK WEIGHT

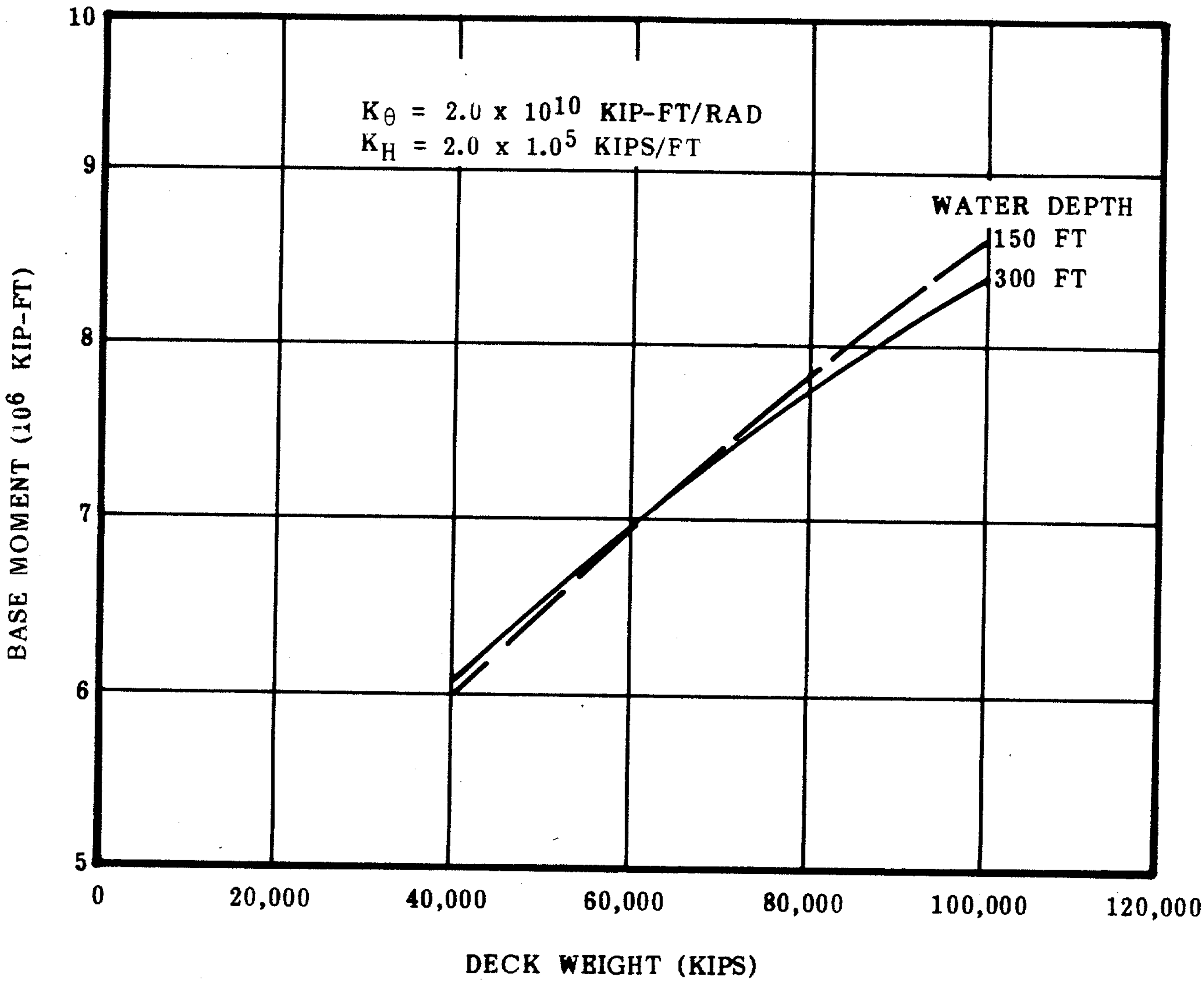


FIGURE 5.1.8 <u>OVERTURNING MOMENT AS A FUNCTION OF DECK WEIGHT</u>

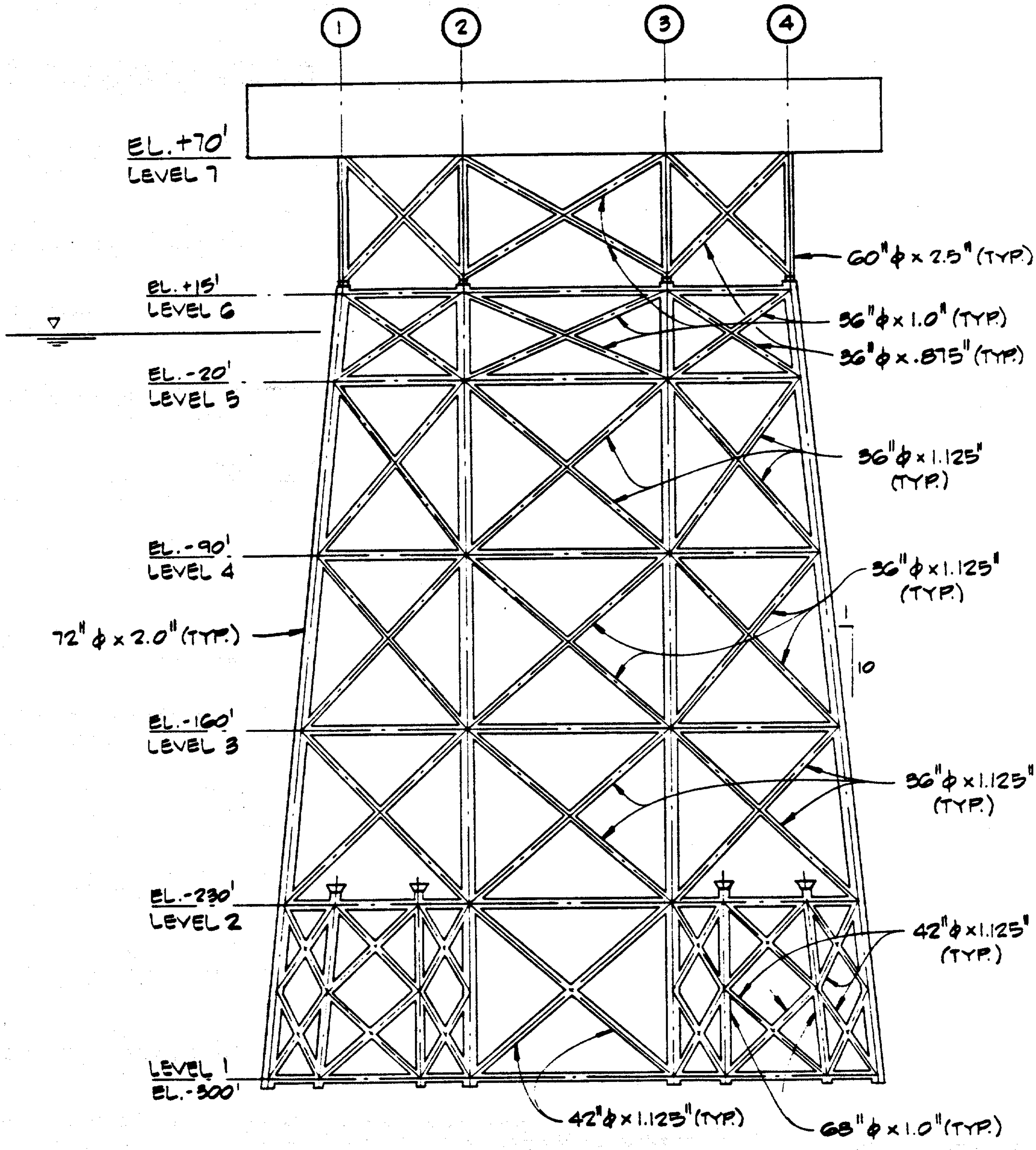


FIGURE 5.2.1 300 FT WATER DEPTH - BASE CASE

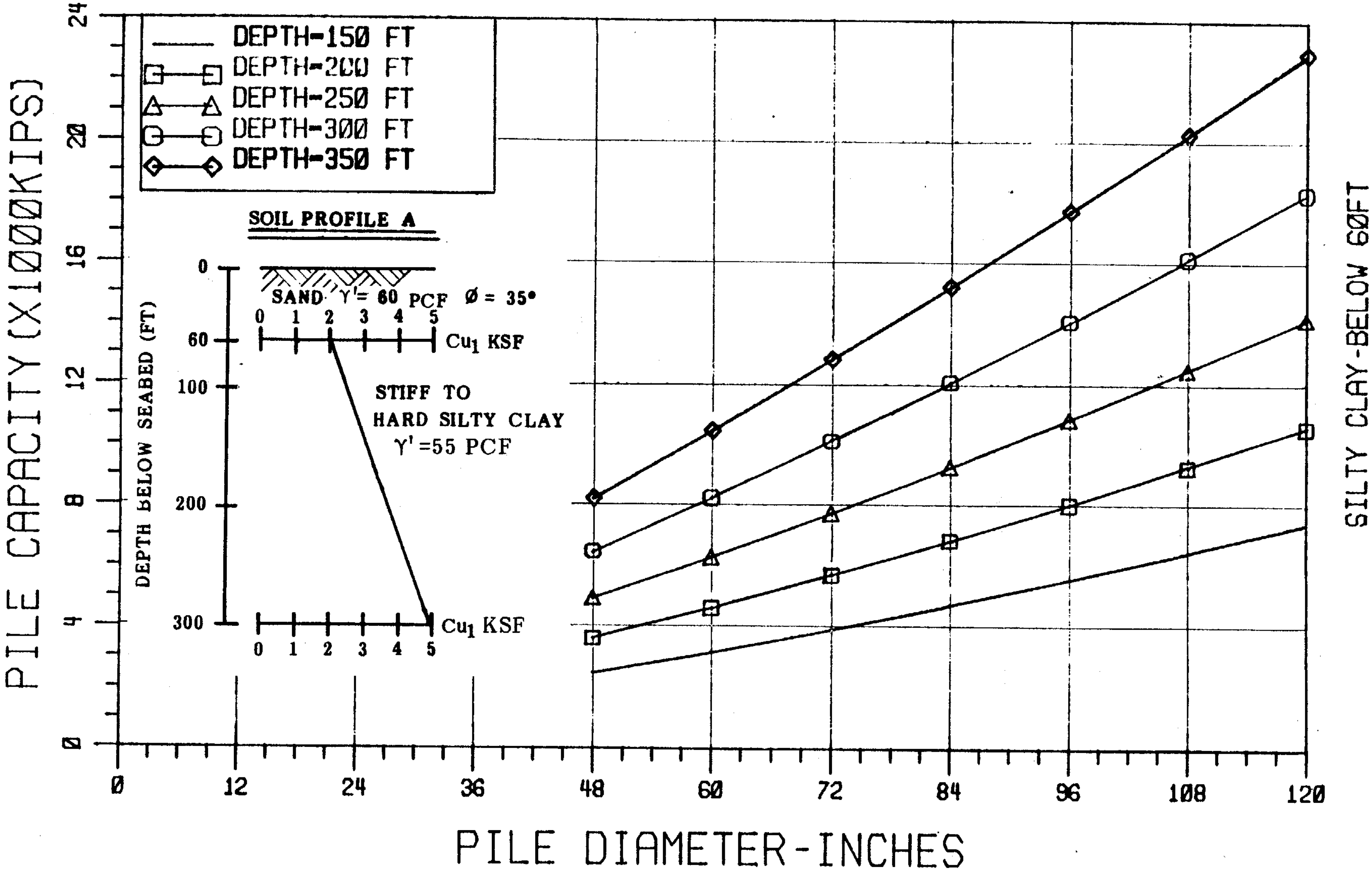


FIGURE 5.2.2 <u>PILE CAPACITY FOR PROFILE A</u>

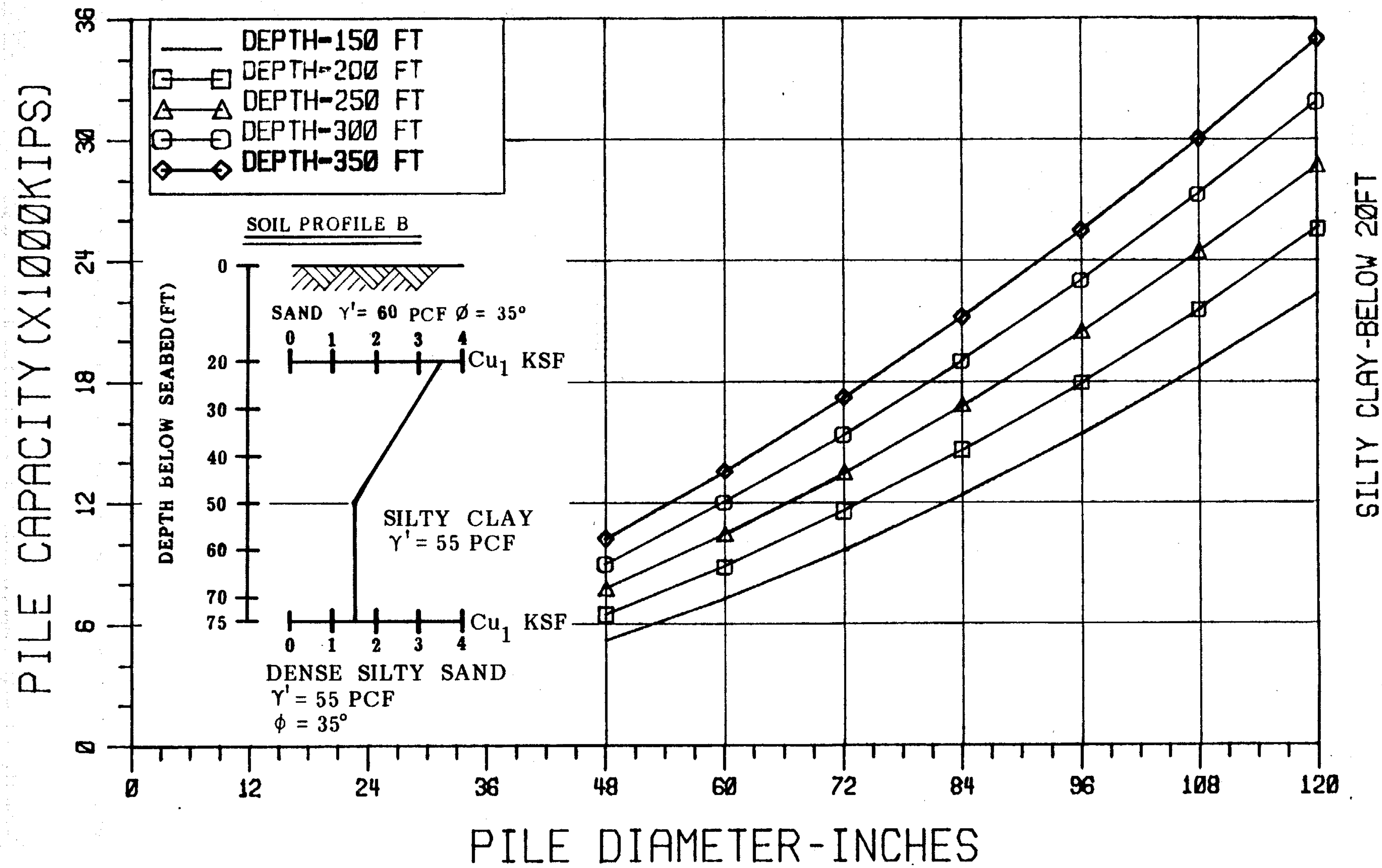


FIGURE 5.2.3 PILE CAPACITY FOR PROFILE B

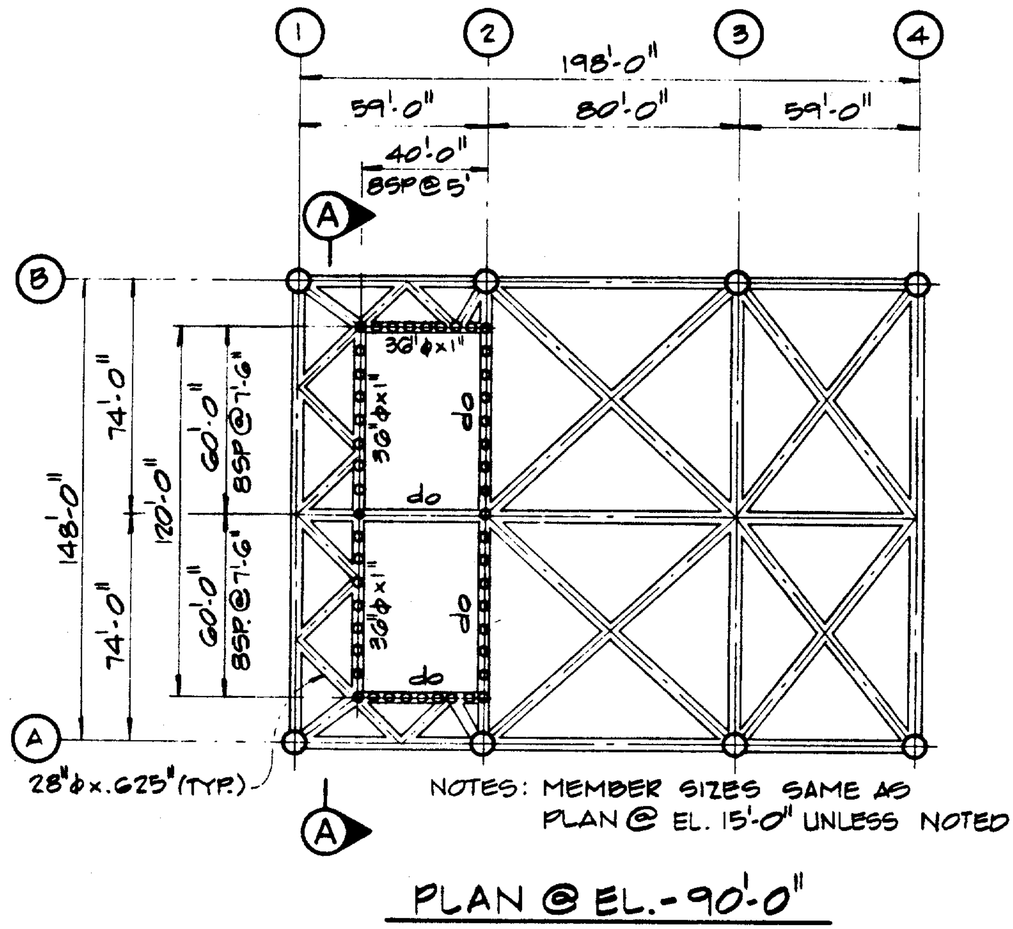


FIGURE 5.2.4 TYPICAL PLAN OF THE CONDUCTOR CAGE PROTECTION SYSTEM

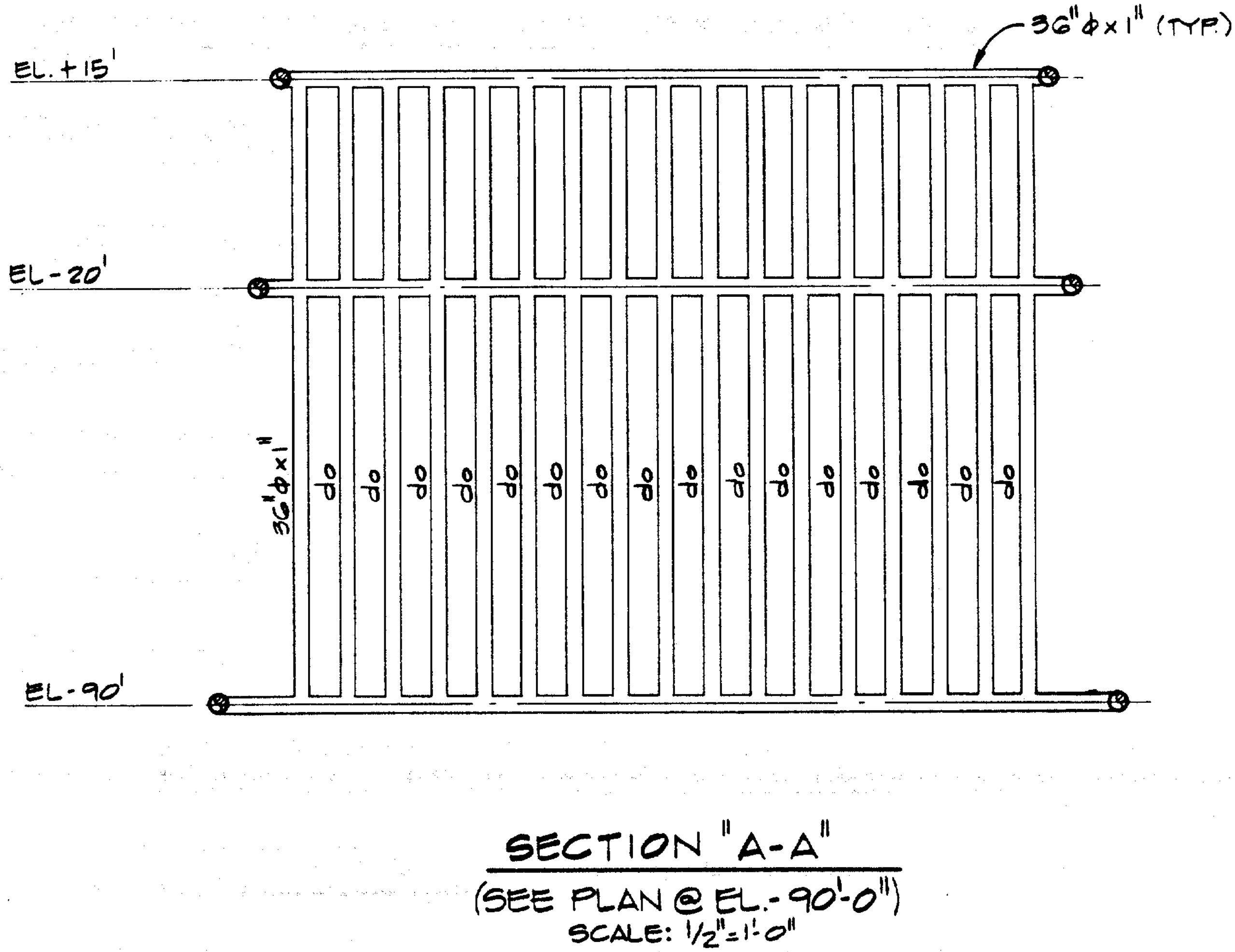
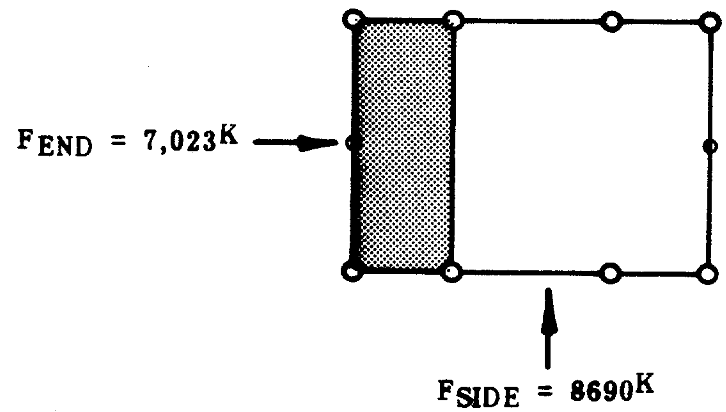


FIGURE 5.2.5 ELEVATION OF THE CONDUCTOR CAGE PROTUCTION SYSTEM



	END	SIDE
LOAD ON LEGS	1656^K	3312^K
LOAD ON BRACING	3918^K	4964^K
LOAD ON ICE CAGE	1449^K	414^K
GLOBAL ICE LOAD	7023^K	8690^K

FIGURE 5.2.6 GLOBAL ICE LOADS ON THE JACKET

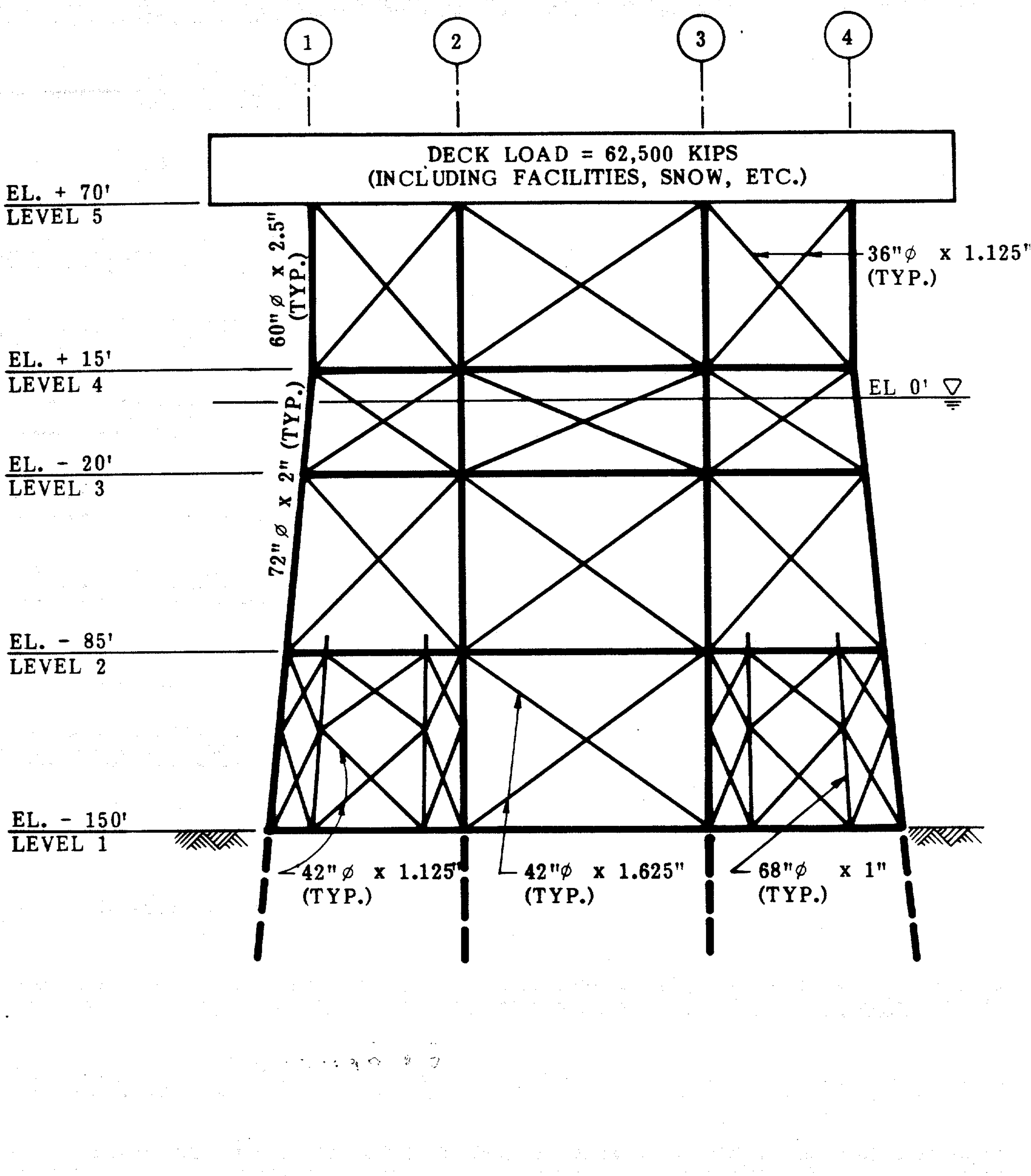
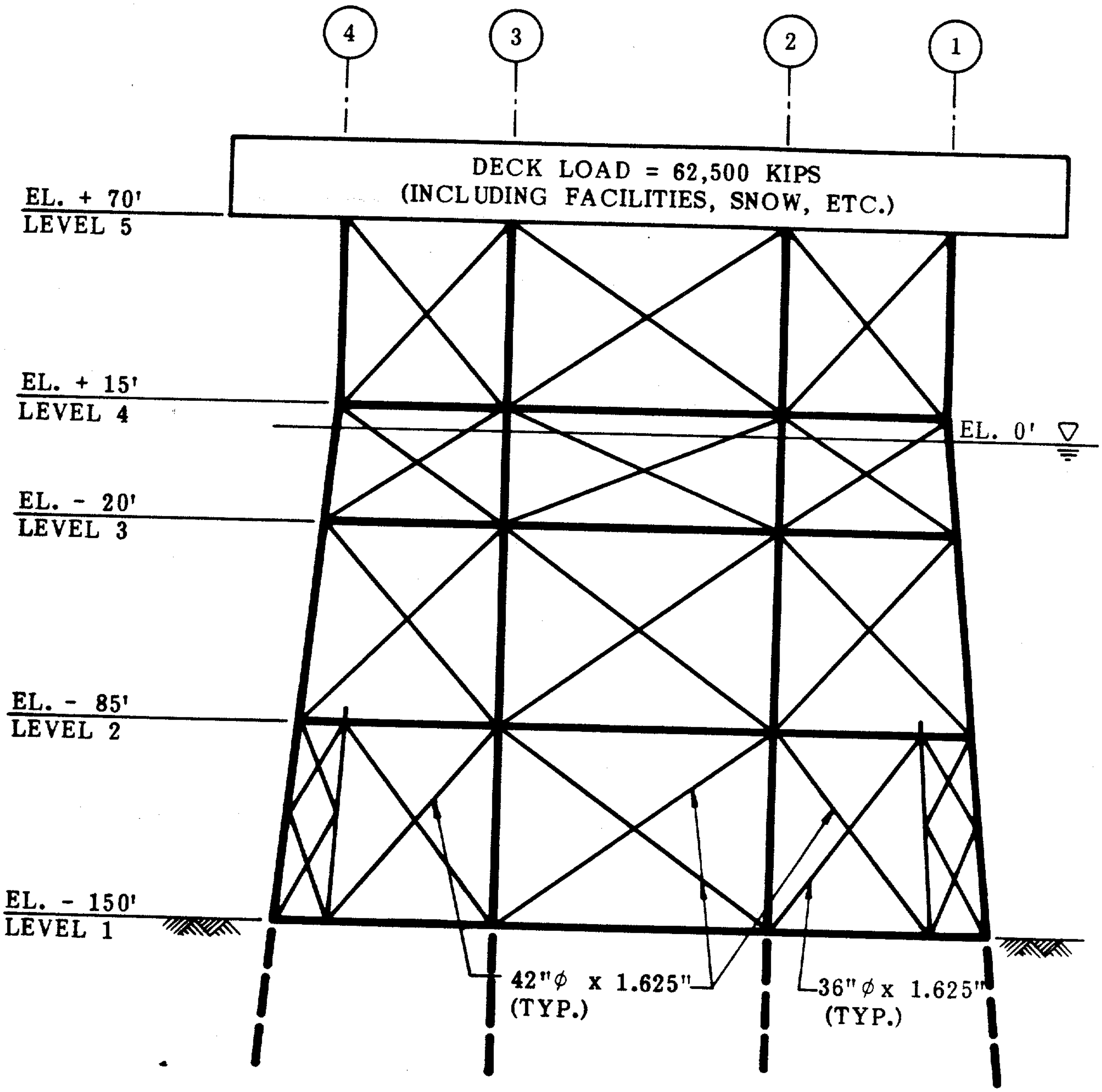


FIGURE 5.2.7 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, ROW A



Note: Bracing is identical to Row A (Figure 5.2.7) unless noted otherwise.

FIGURE 5.2.8 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, ROW B

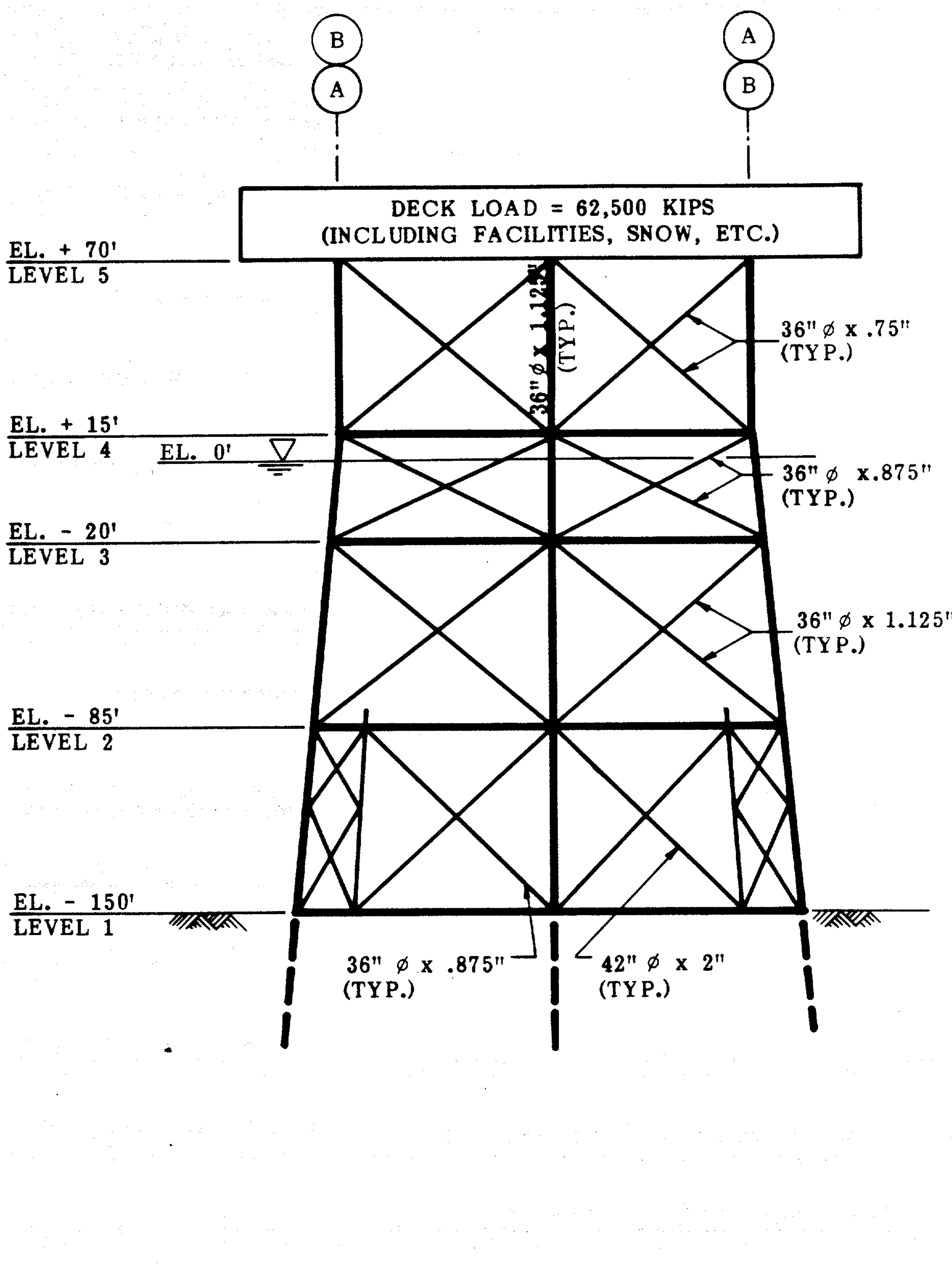


FIGURE 5.2.9 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, ROWS 1 AND 4

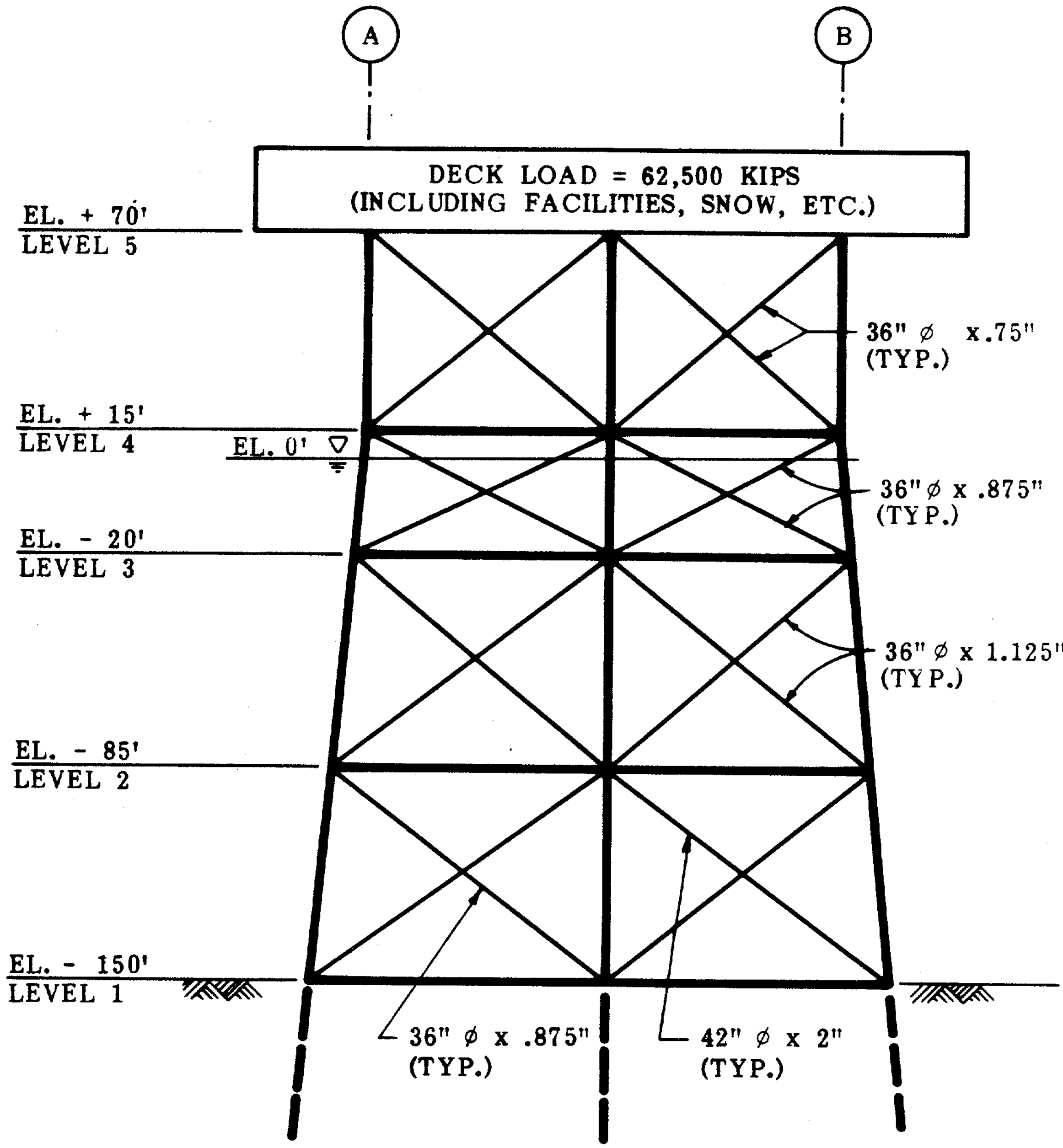


FIGURE 5.2.10 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, ROWS 2 AND 3

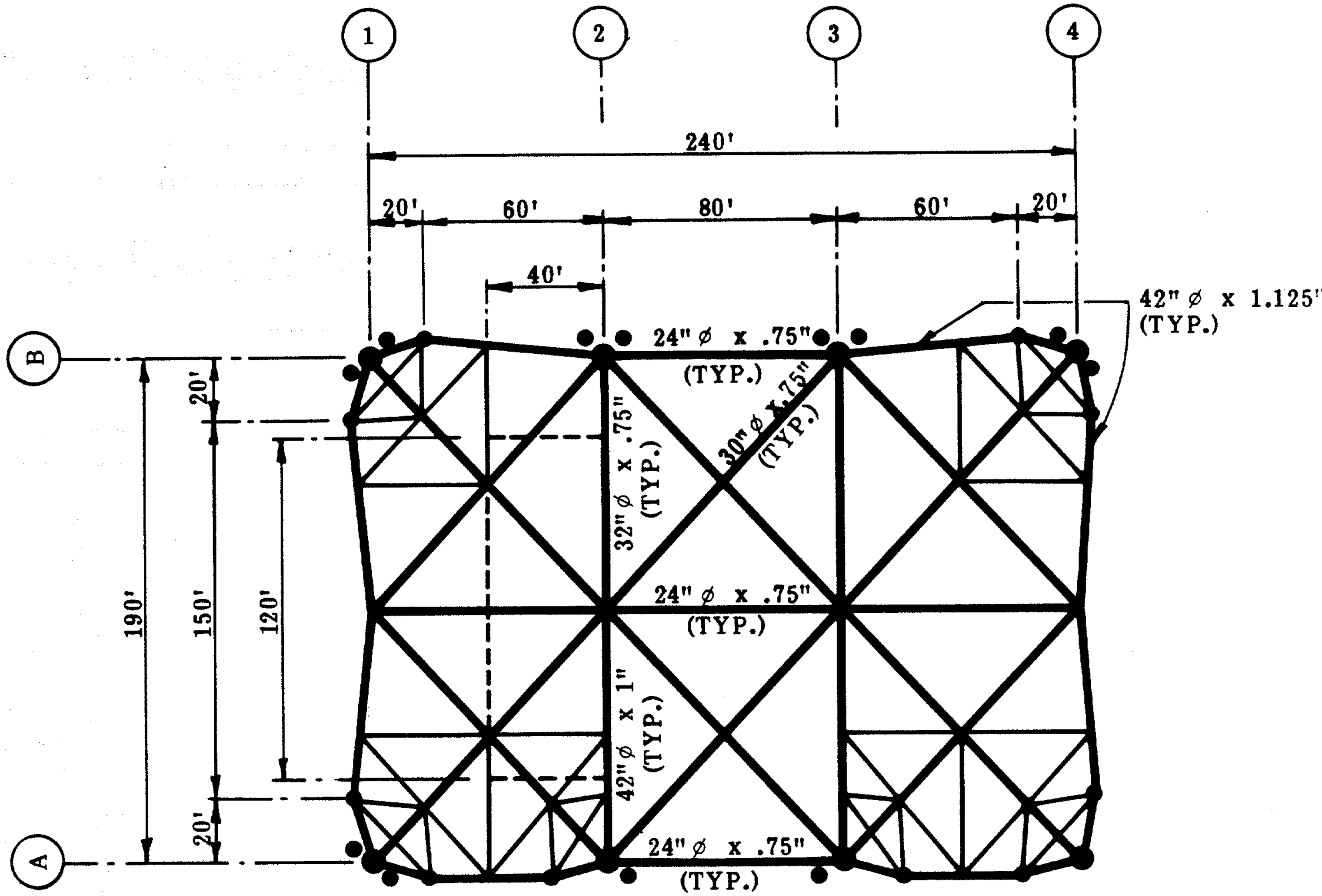
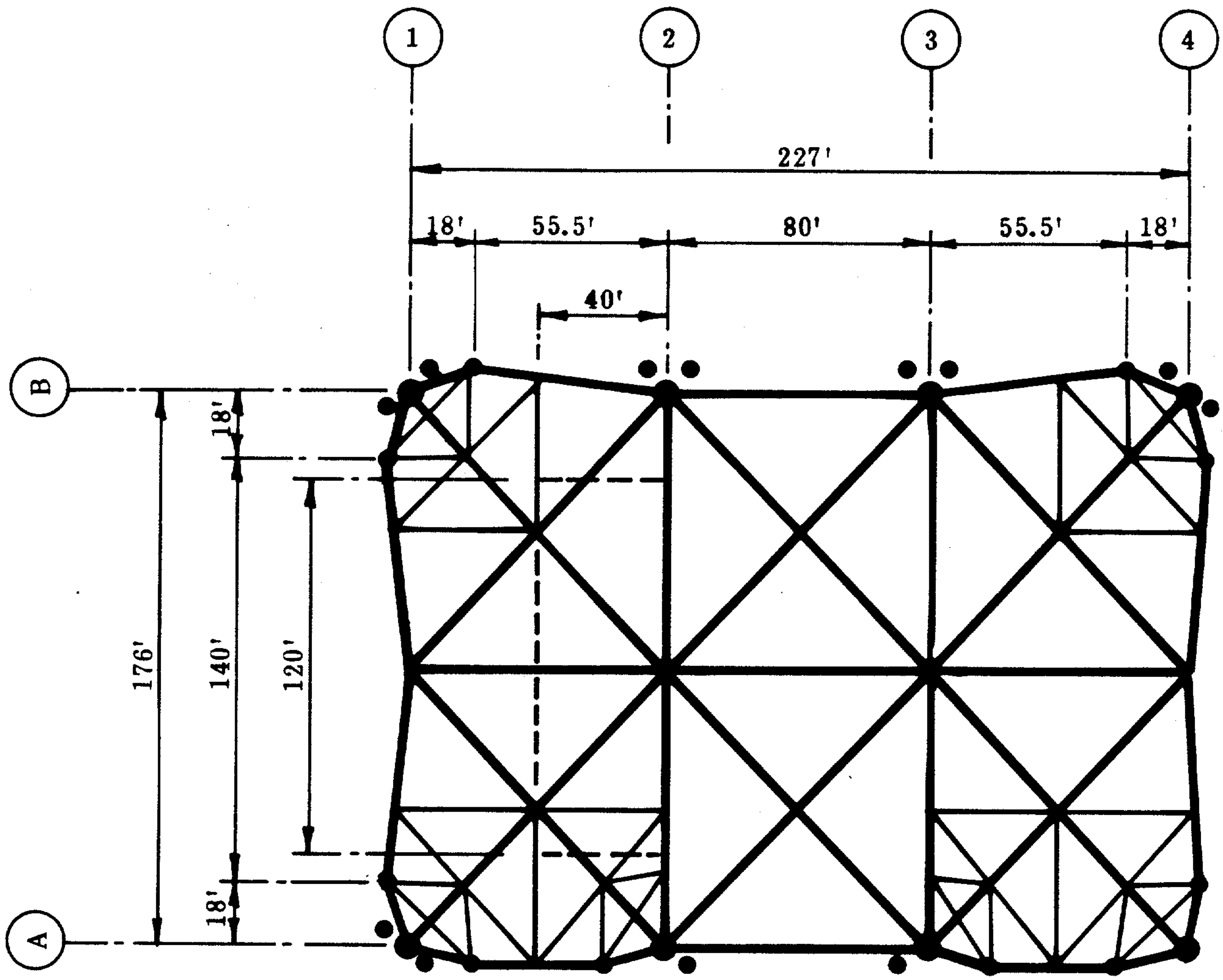


FIGURE 5.2.11 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, LEVEL 1 @ ELEVATION -150' FT



NOTE: BRACING IS IDENTICAL TO LEVEL 1 (FIGURE 5.2.11)

FIGURE 5.2.12 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM LEVEL, 2 @ ELEVATION -85 FT

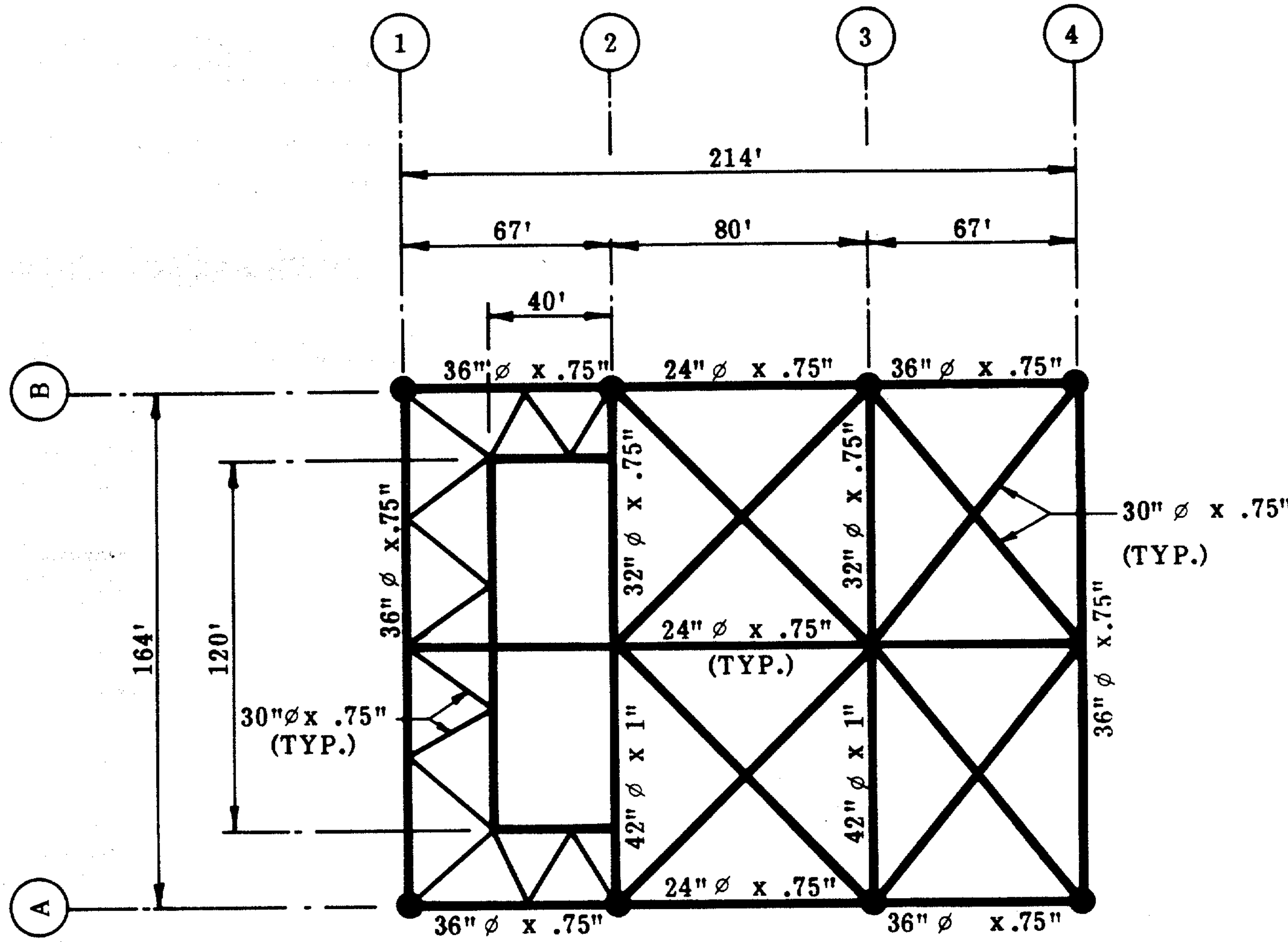


FIGURE 5.2.13 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, LEVEL 3 @ ELEVATION - 20 FT

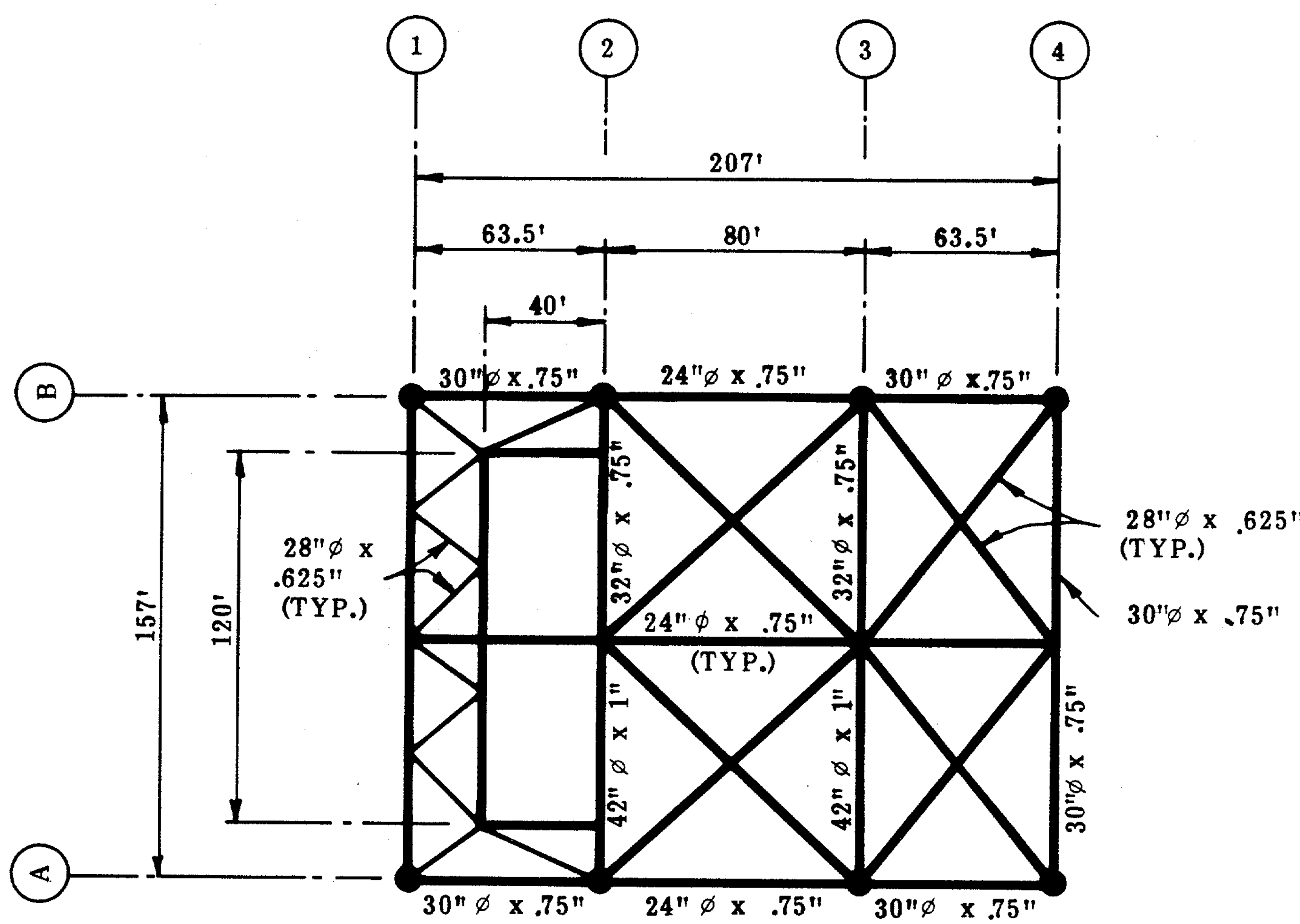


FIGURE 5.2.14 SINGLE PIECE, 150 FT WATER DEPTH PLATFORM, LEVEL 2 @ ELEVATION - 85 FT

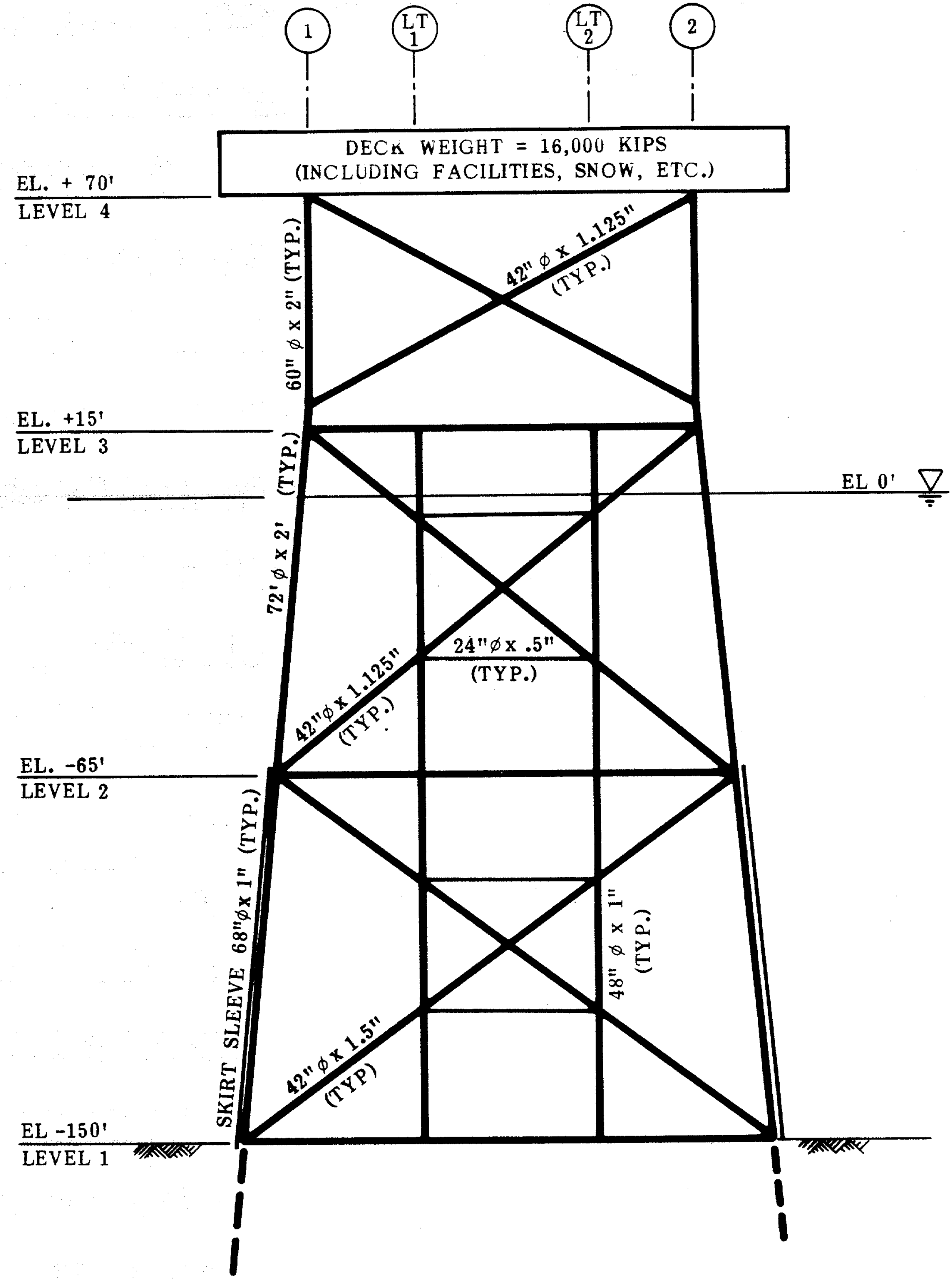


FIGURE 5.2.15 **MULTIPLE JACKET CASE, 150 FT WATER DEPTH, ROW A**

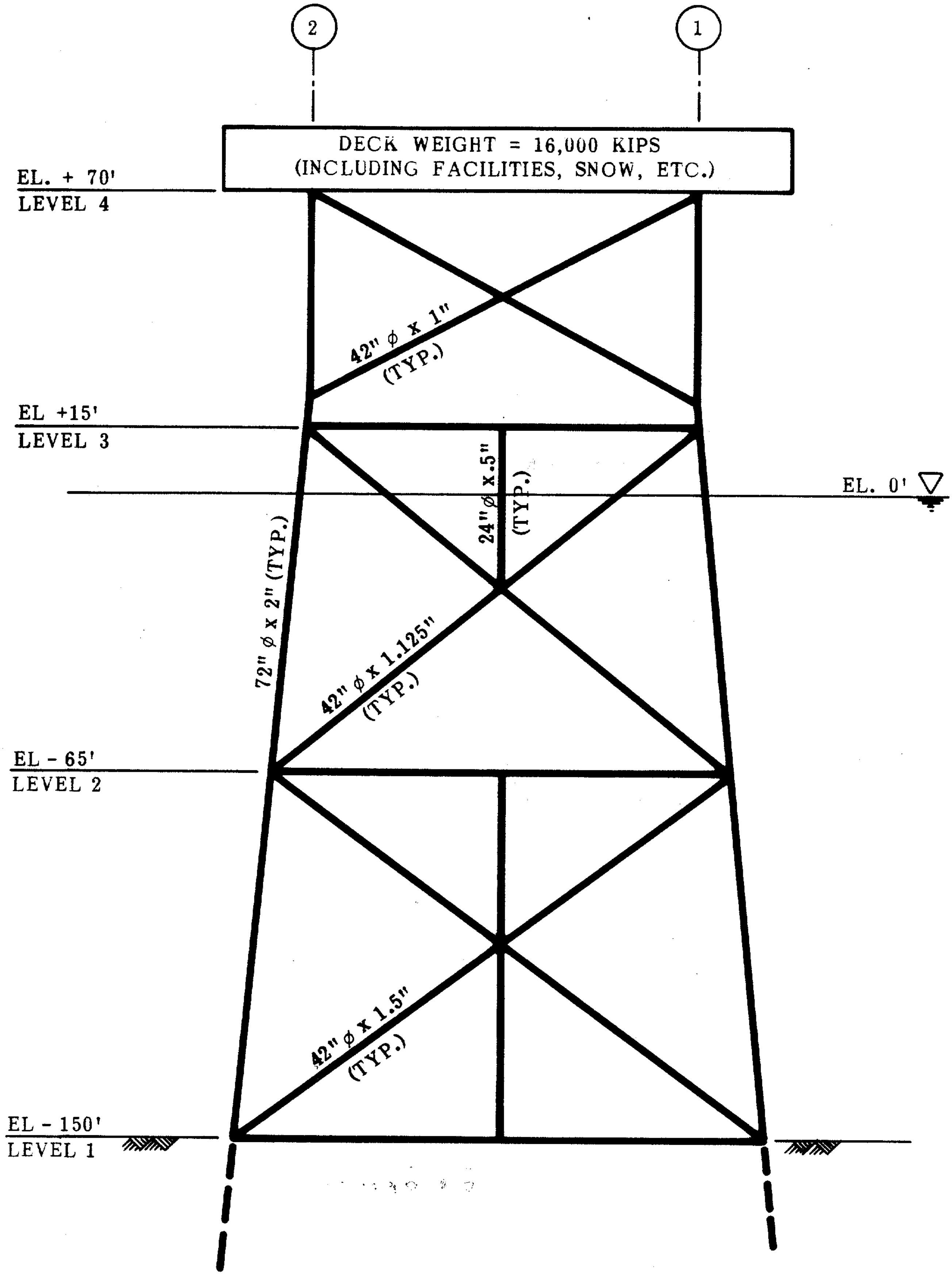


FIGURE 5.2.16 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, ROW B

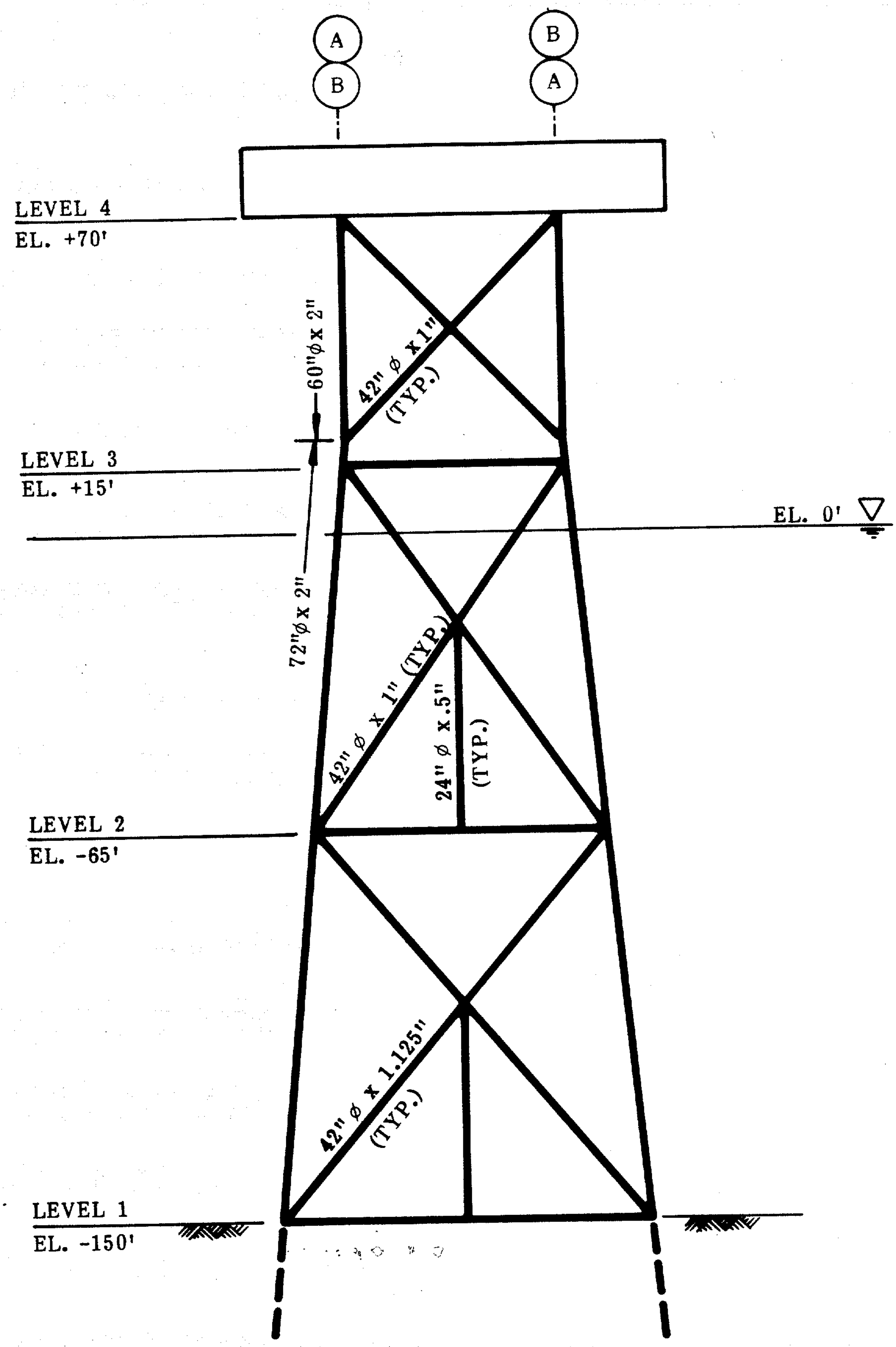


FIGURE 5.2.17 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, ROWS 1 AND 2

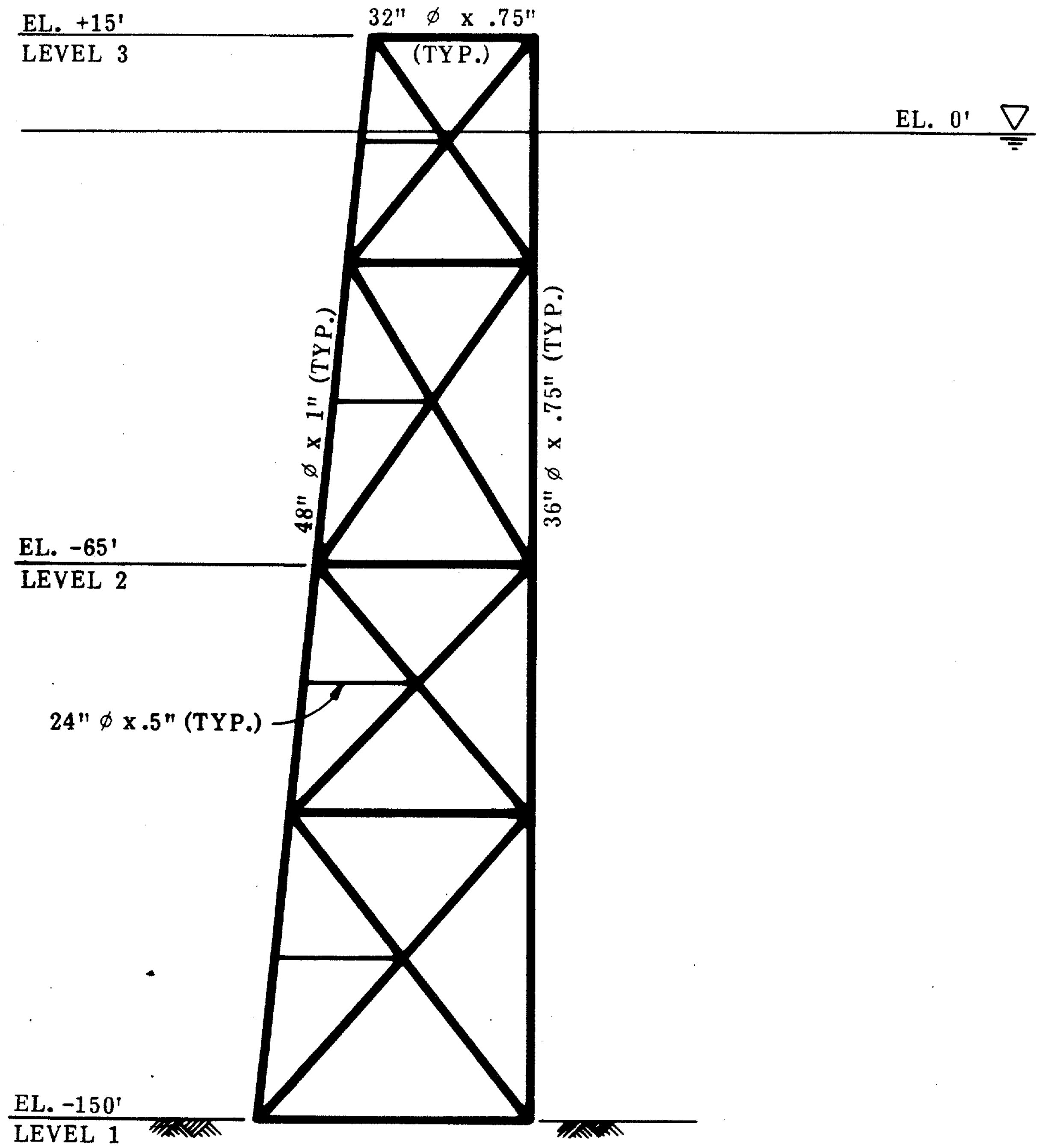


FIGURE 5.2.18 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, LAUNCH TRUSS

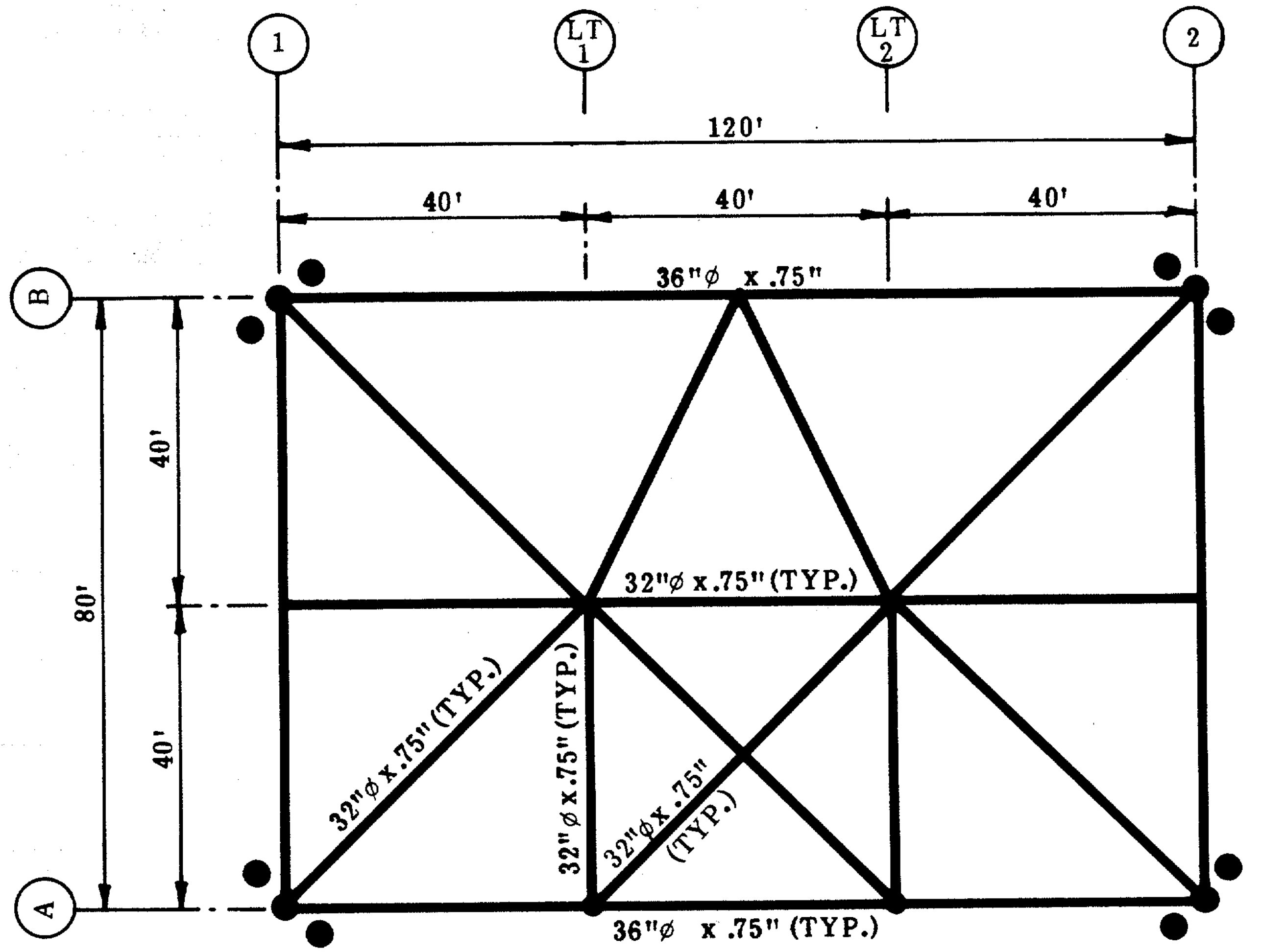
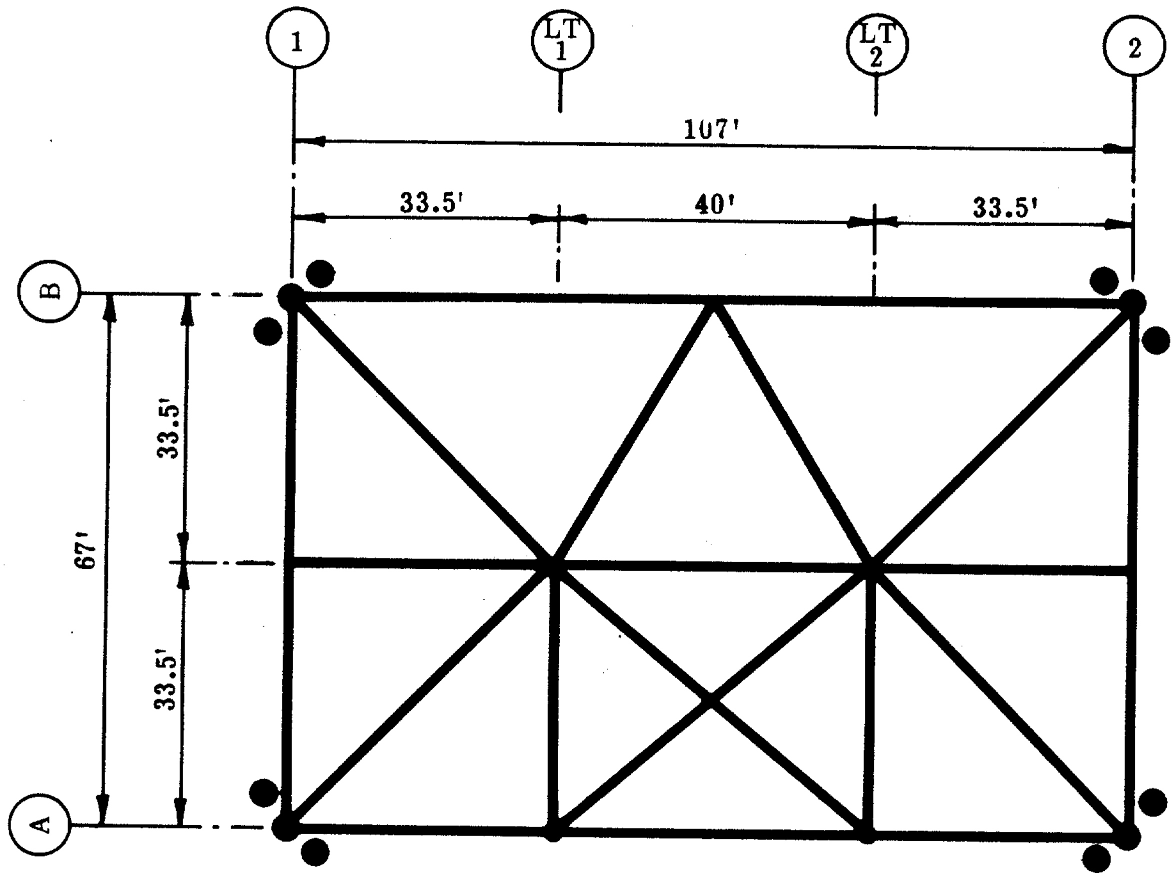
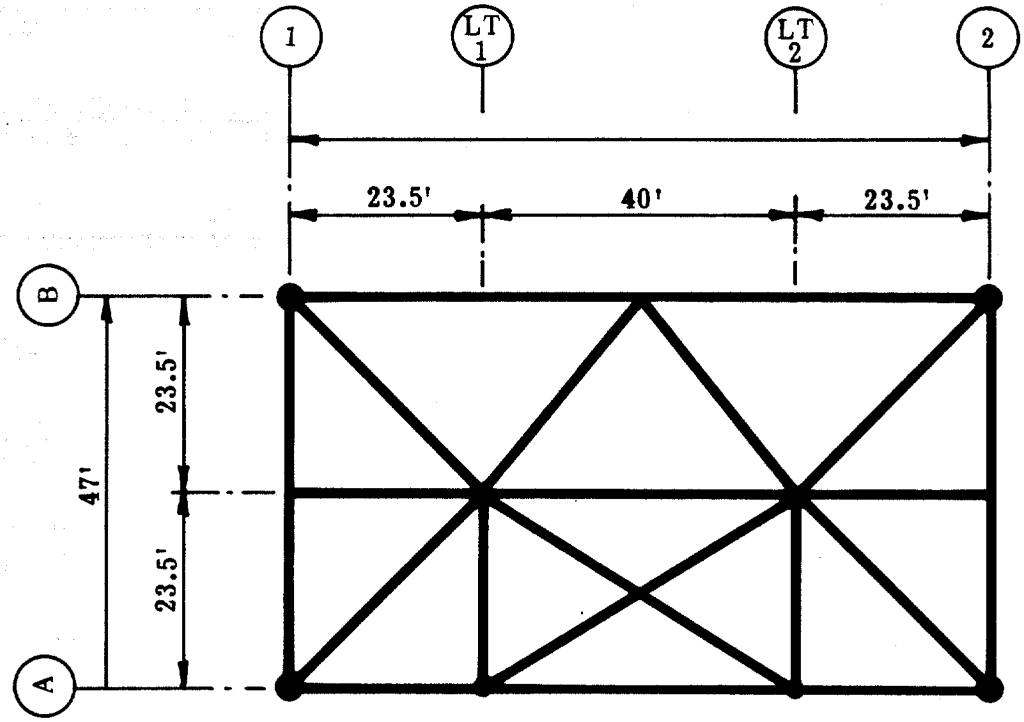


FIGURE 5.2.19 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, LEVEL 1 @ ELEVATION -150 FT



NOTE: BRACING IS IDENTICAL TO LEVEL 1 (FIGURE 5.2.19)

FIGURE 5.2.20 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, LEVEL 2 @ ELEVATION -65 FT



NOTE: BRACING IS IDENTICAL TO LEVEL 1 (FIGURE 5.2.19)

FIGURE 5.2.21 MULTIPLE JACKET CASE, 150 FT WATER DEPTH, LEVEL 3 @ ELEVATION +15 FT

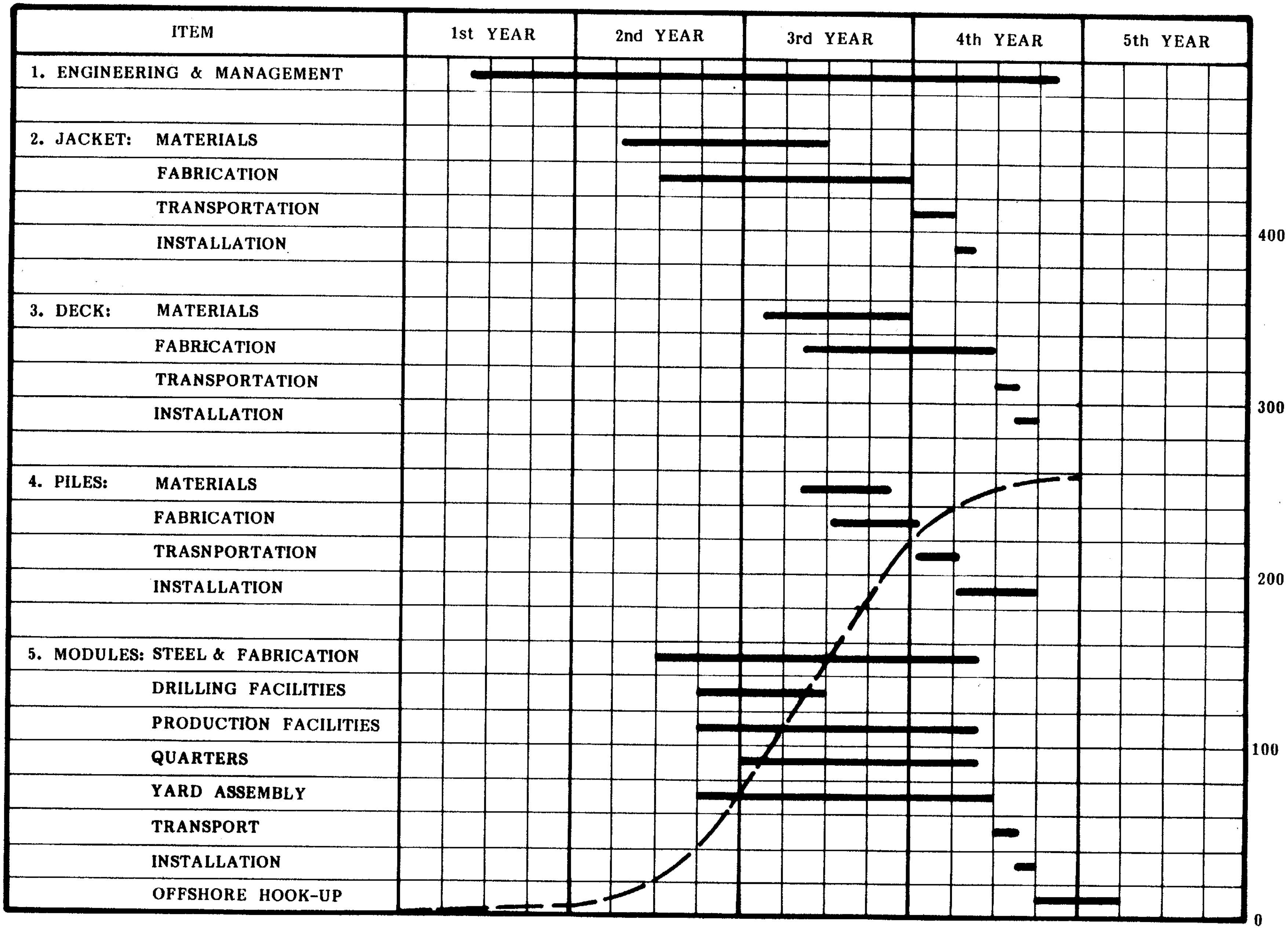


FIGURE 5.3.1 FABRICATION AND INSTALLATION SCHEDULE AND CASH FLOW SUMMARY FOR THE PILED STEEL JACKET IN THE 300 FT WATER DEPTH AND 100,000 BOPD

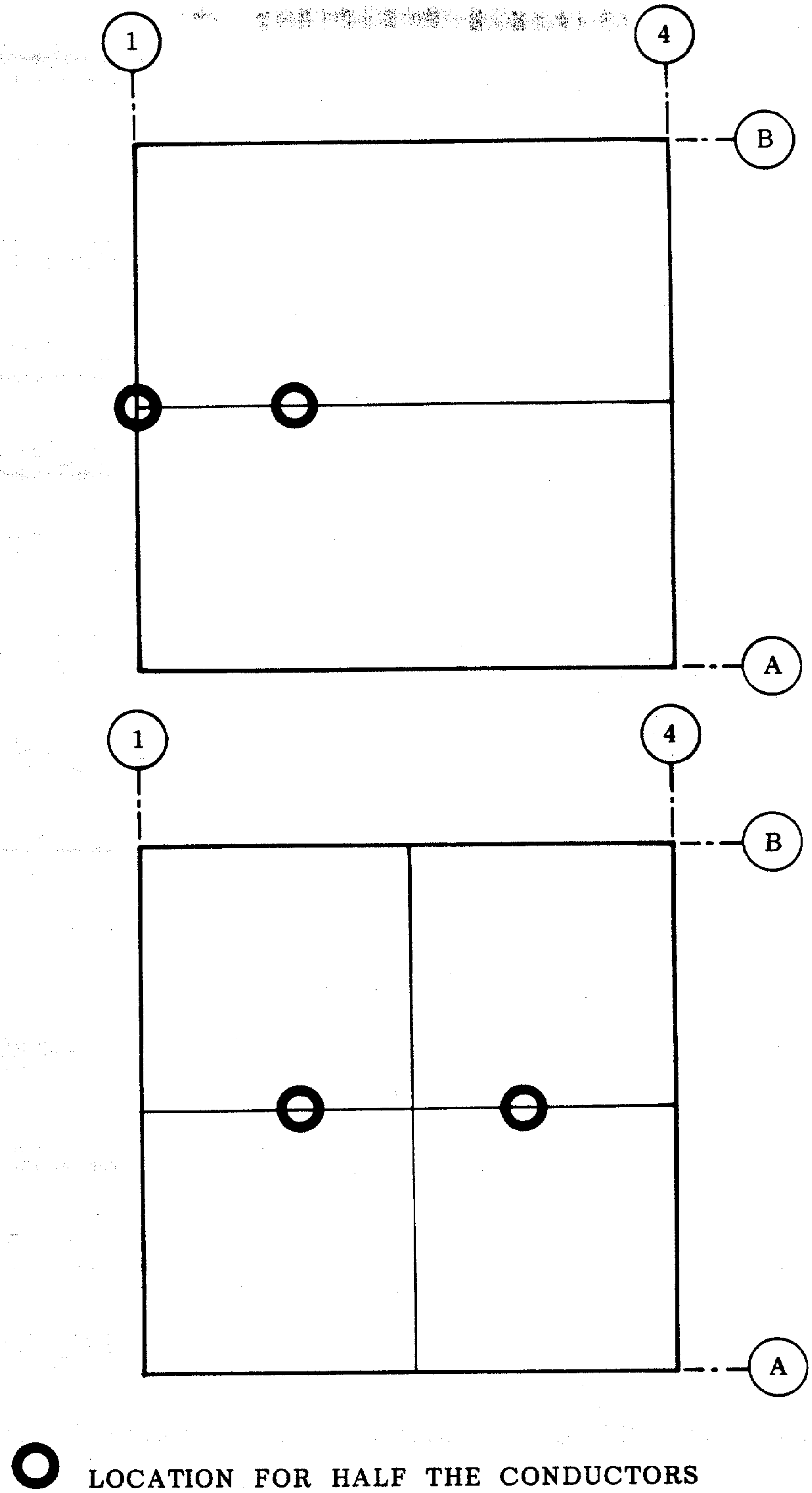


FIGURE 5.4.1 LOCATIONS CONSIDERED FOR THE CONDUCTORS IN THE ANALYSIS

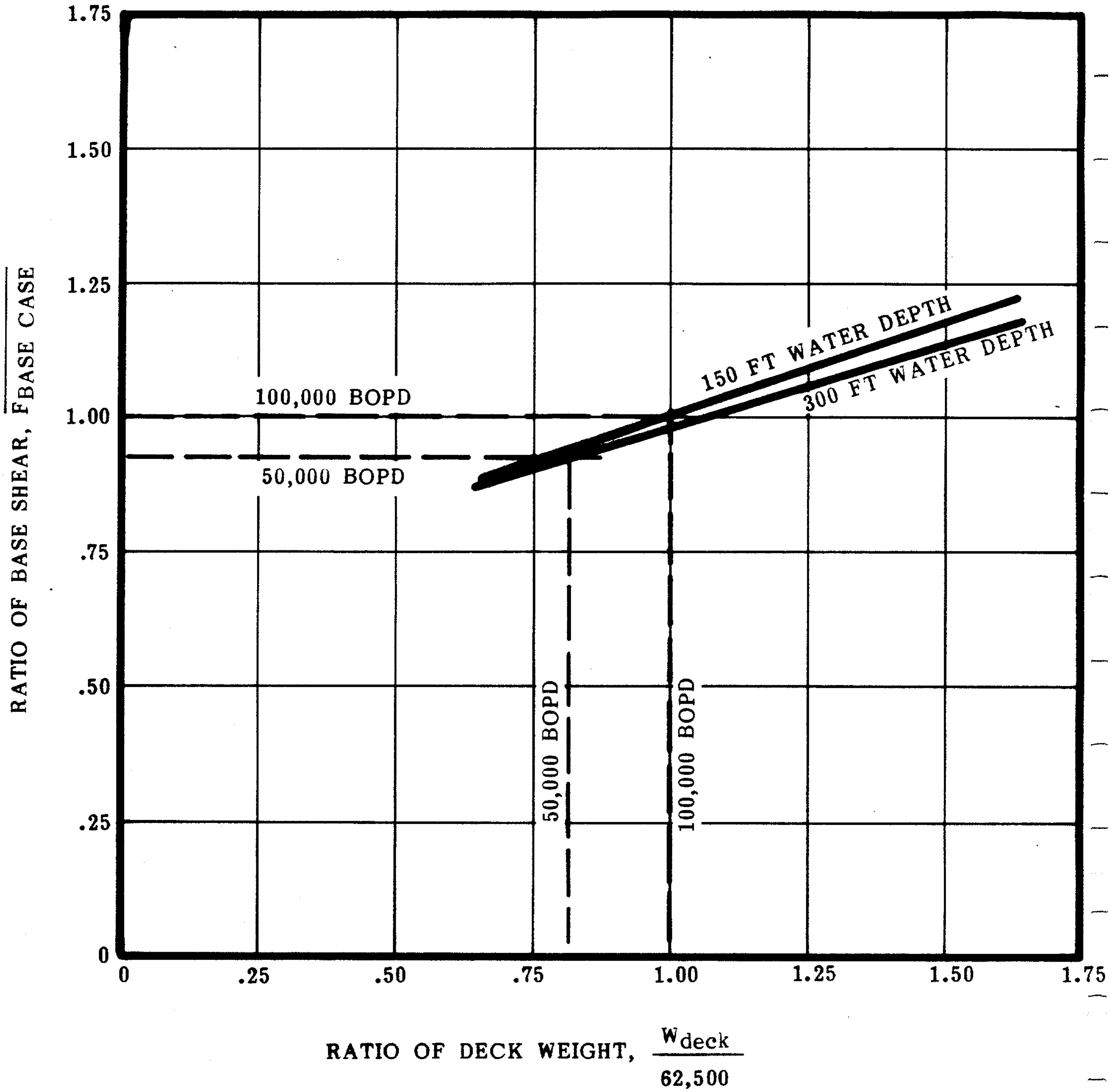


FIGURE 5.4.2 VARIATION IN BASE SHEAR WITH DECK WEIGHT

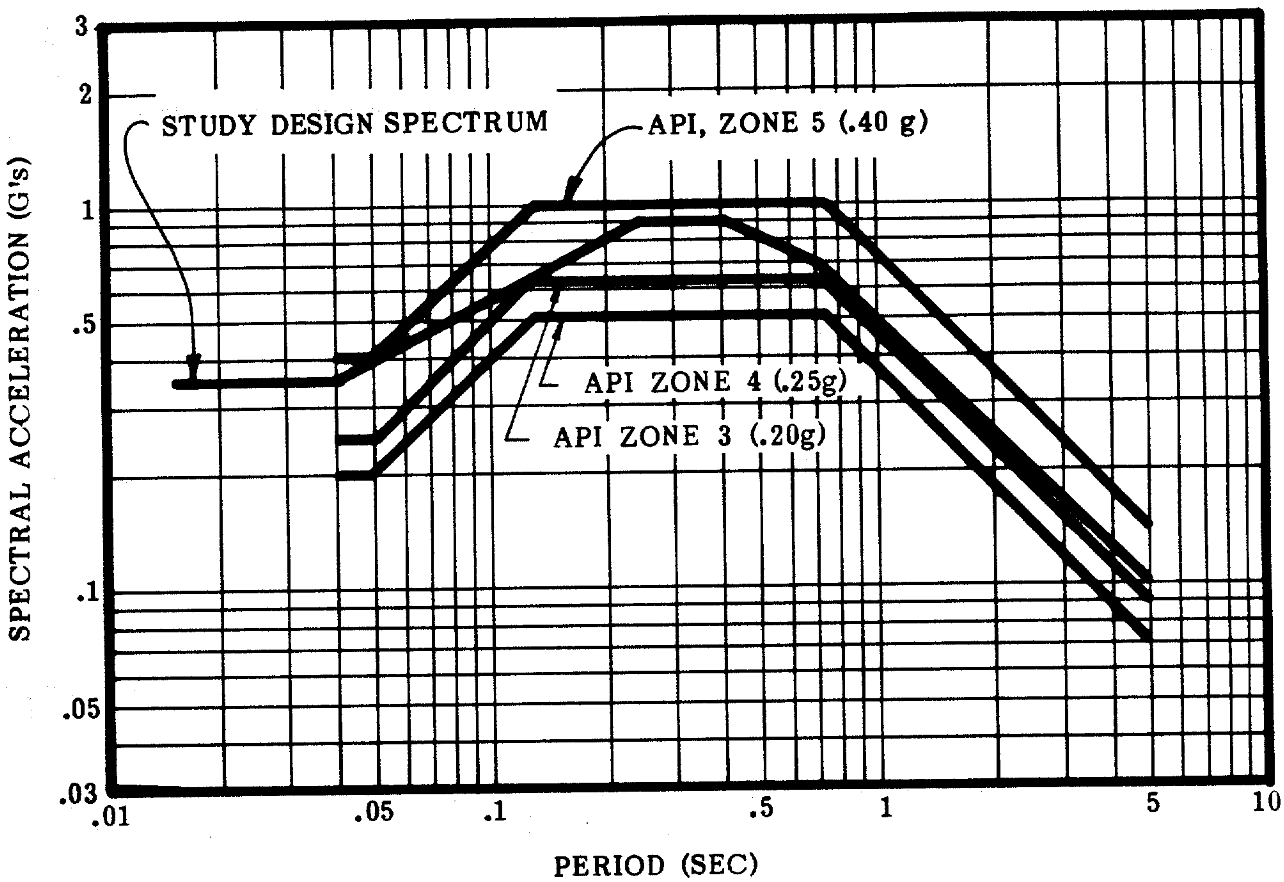


FIGURE 5.4.3 <u>COMPARRISON OF STYDY DESIGN SPECTRUM AND API</u>

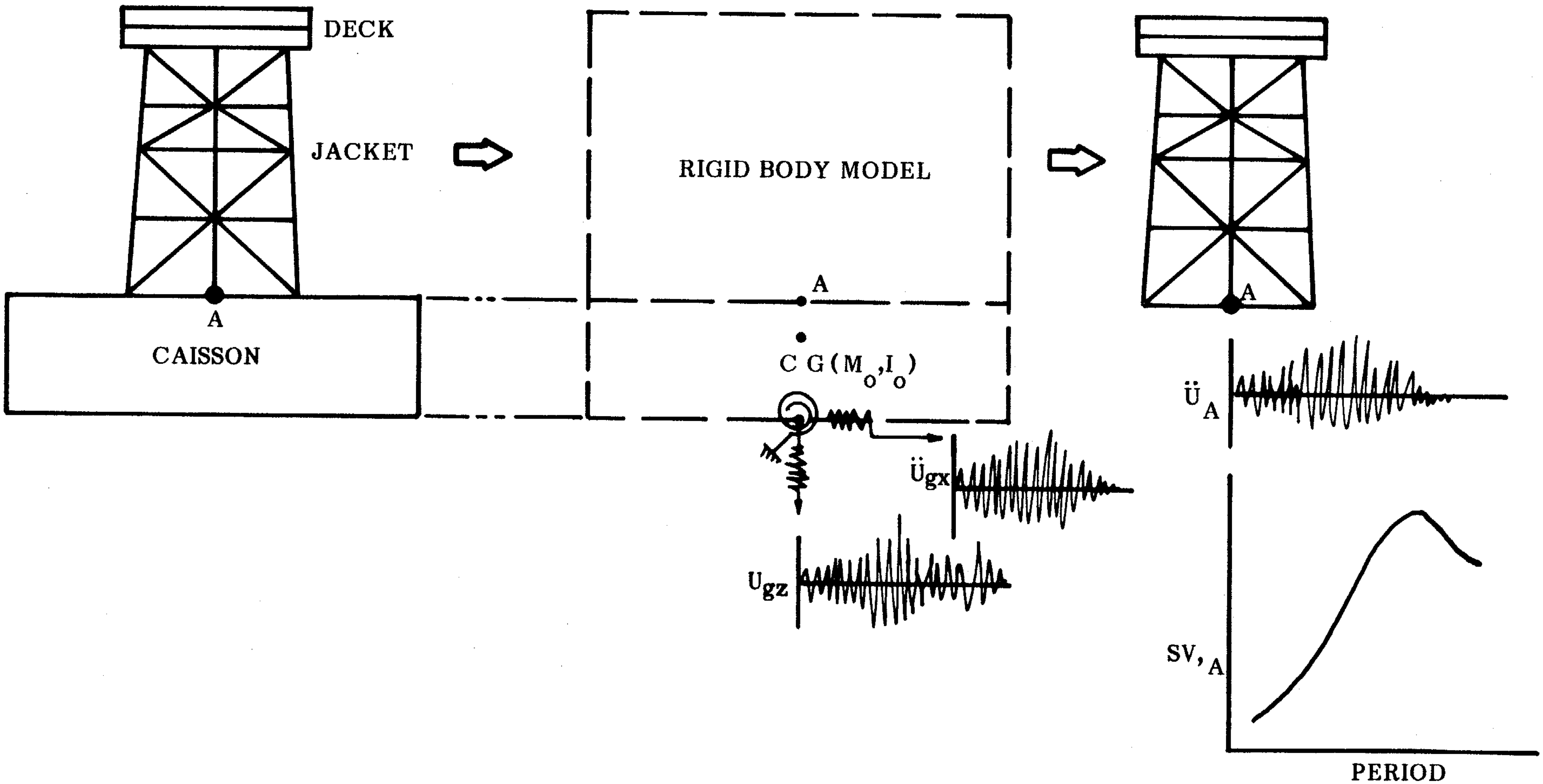


FIGURE 6.1.1 SEISMIC ANALYSIS METHOD FOR HYBRID AND CONCRETE GRAVITY STRUCTURES

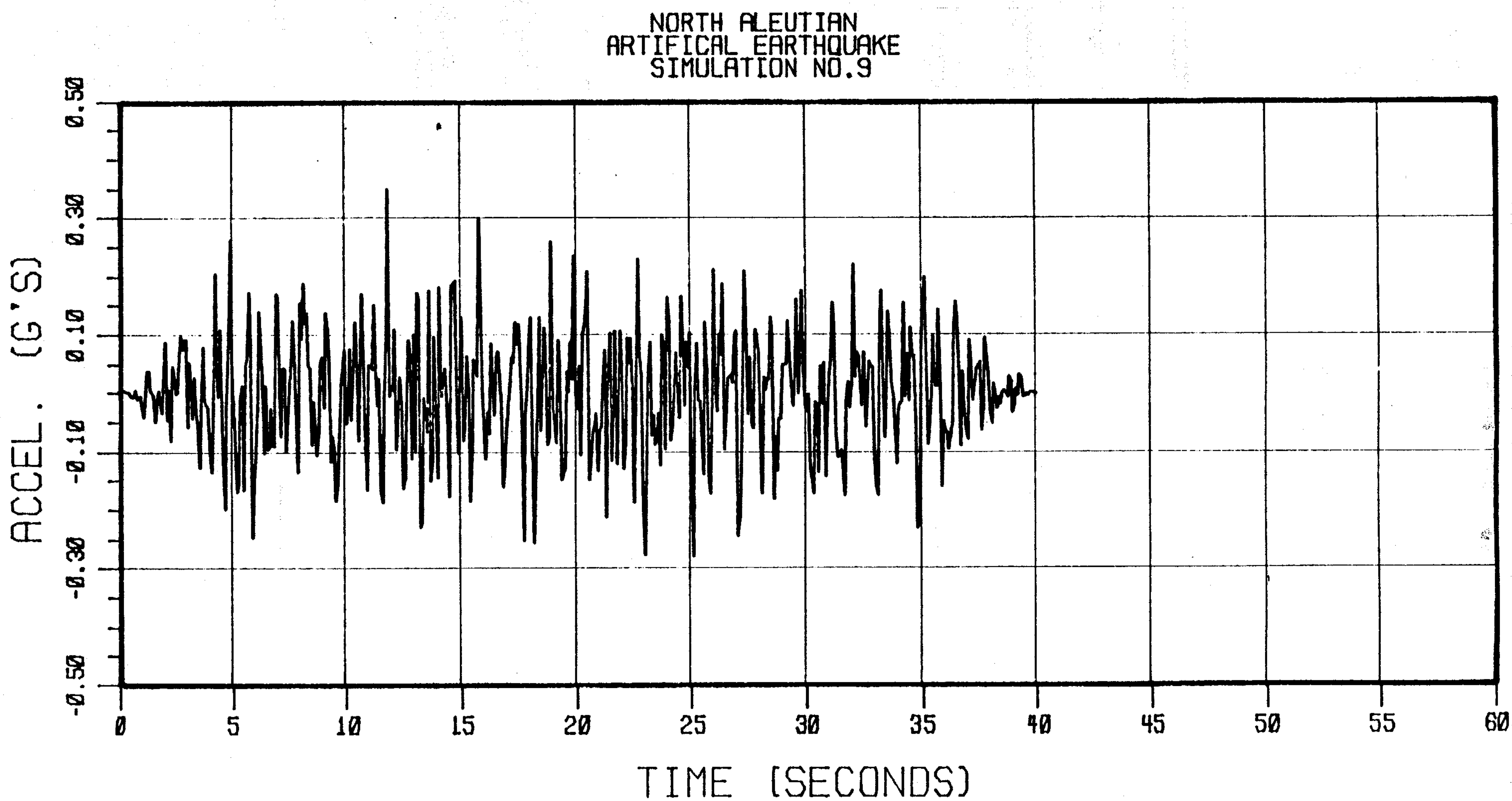


FIGURE 6.1.2 TYPICAL ARTIFICIAL EARTHQUAKE GENERATED TO MATCH THE DESIGN SPECTRUM

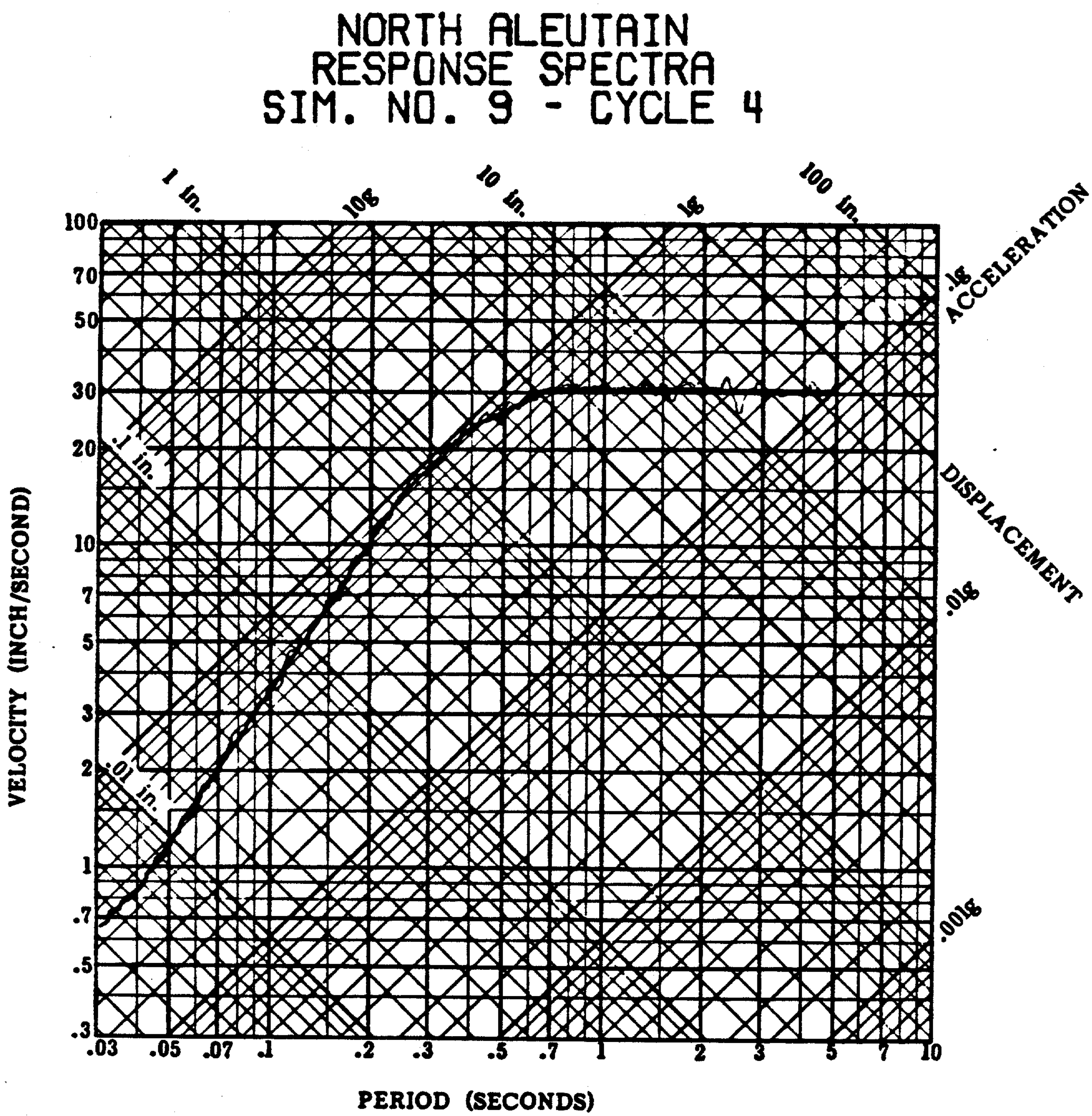
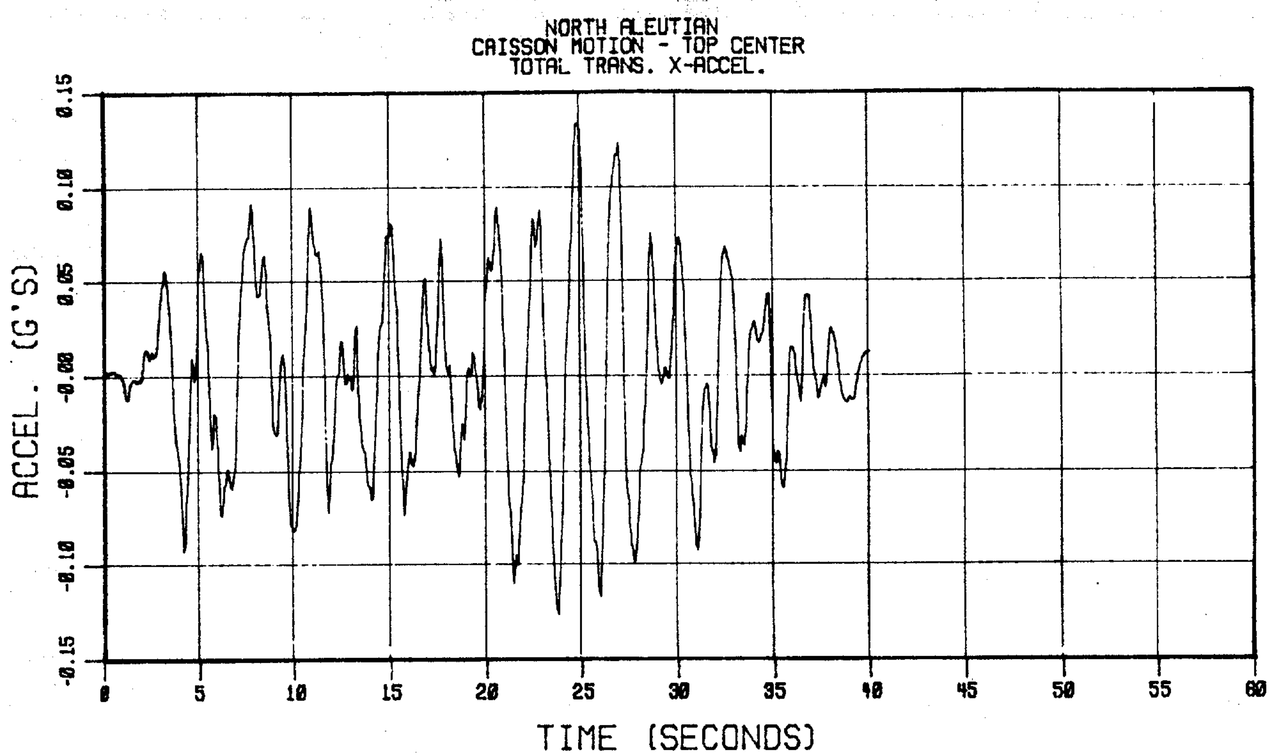


FIGURE 6.1.3 COMPARISON OF TARGET DESIGN SPECTRUM AND SPECTRUM PRODUCED BY THE ARTIFICIAL EARTHQUAKE



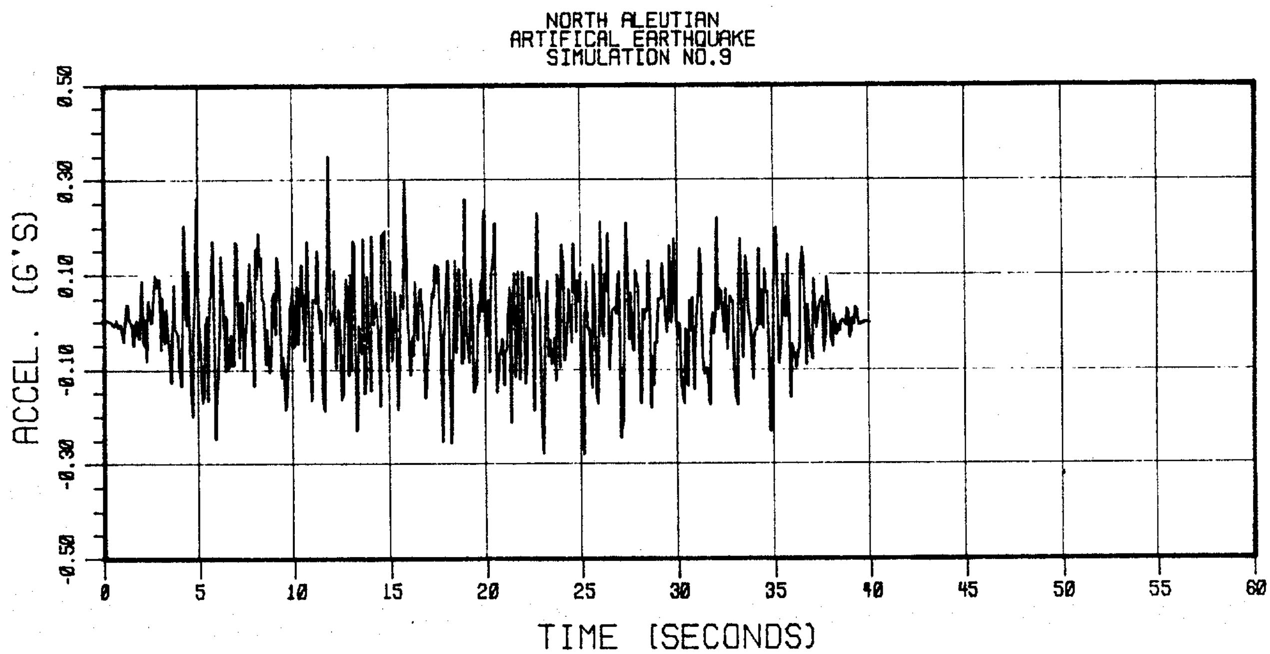


FIGURE 6.1.4 COMPARISON OF INPUT GROUND MOTION AND MOTION AT THE TOP OF THE BASE CAISSON

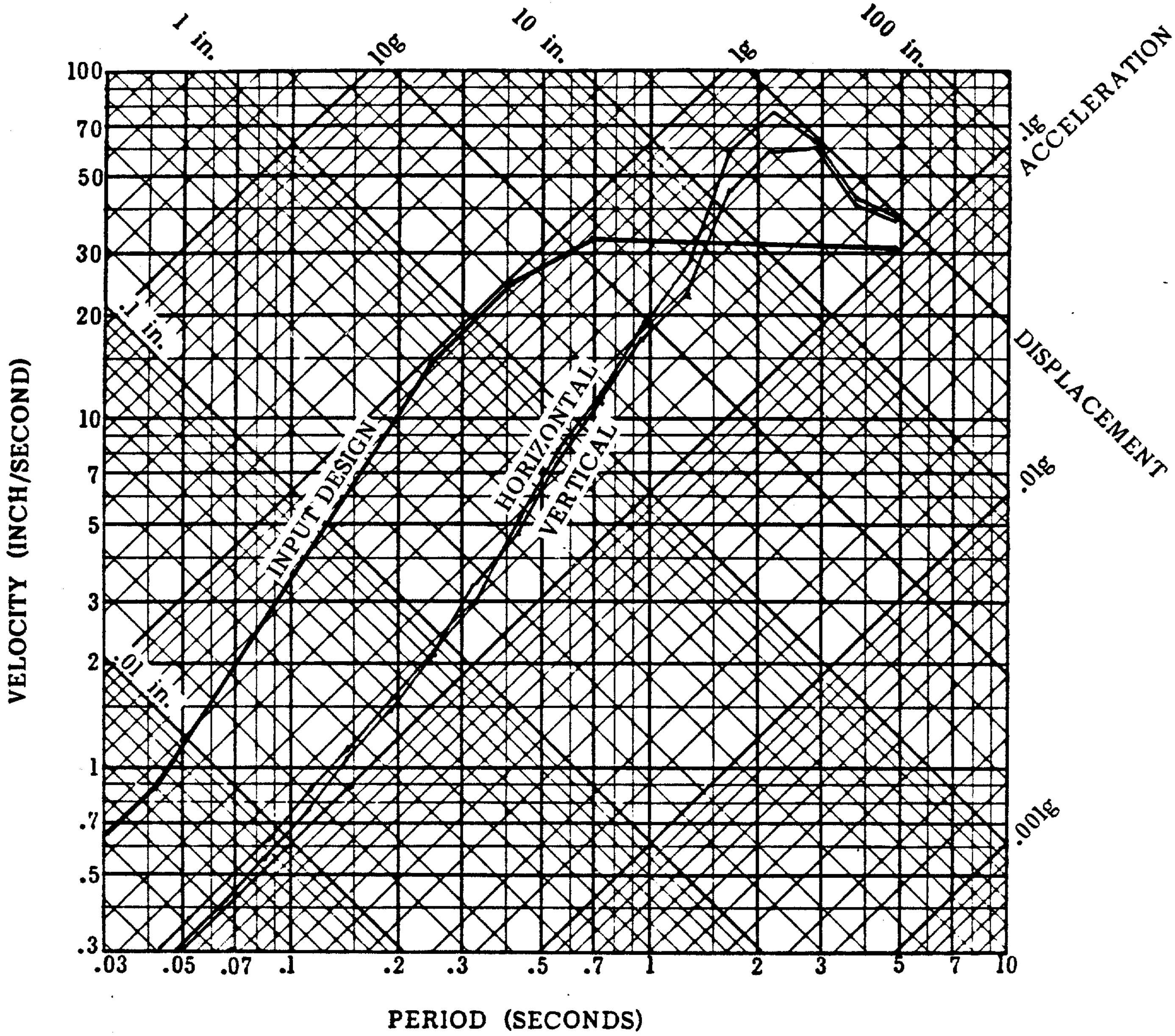


FIGURE 6.1.5 COMPARISON OF INPUT DESIGN SPECTRUM AND VERTICAL AND HORIZONTAL SPECTRUM AT THE CAISSON TOP

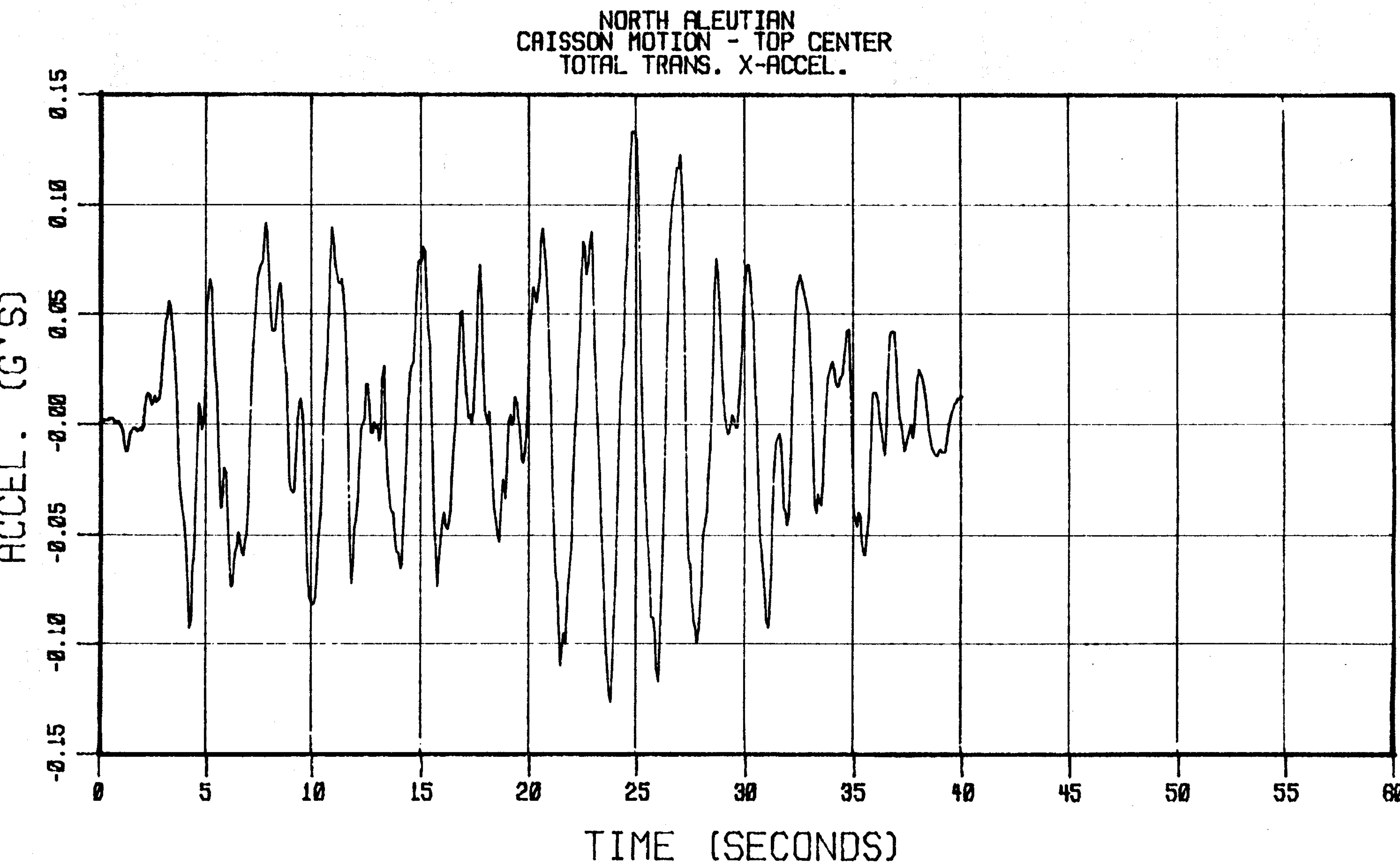


FIGURE 6.2.1 HORIZONTAL ACCELERATION AT THE CAISSON TOP CENTER

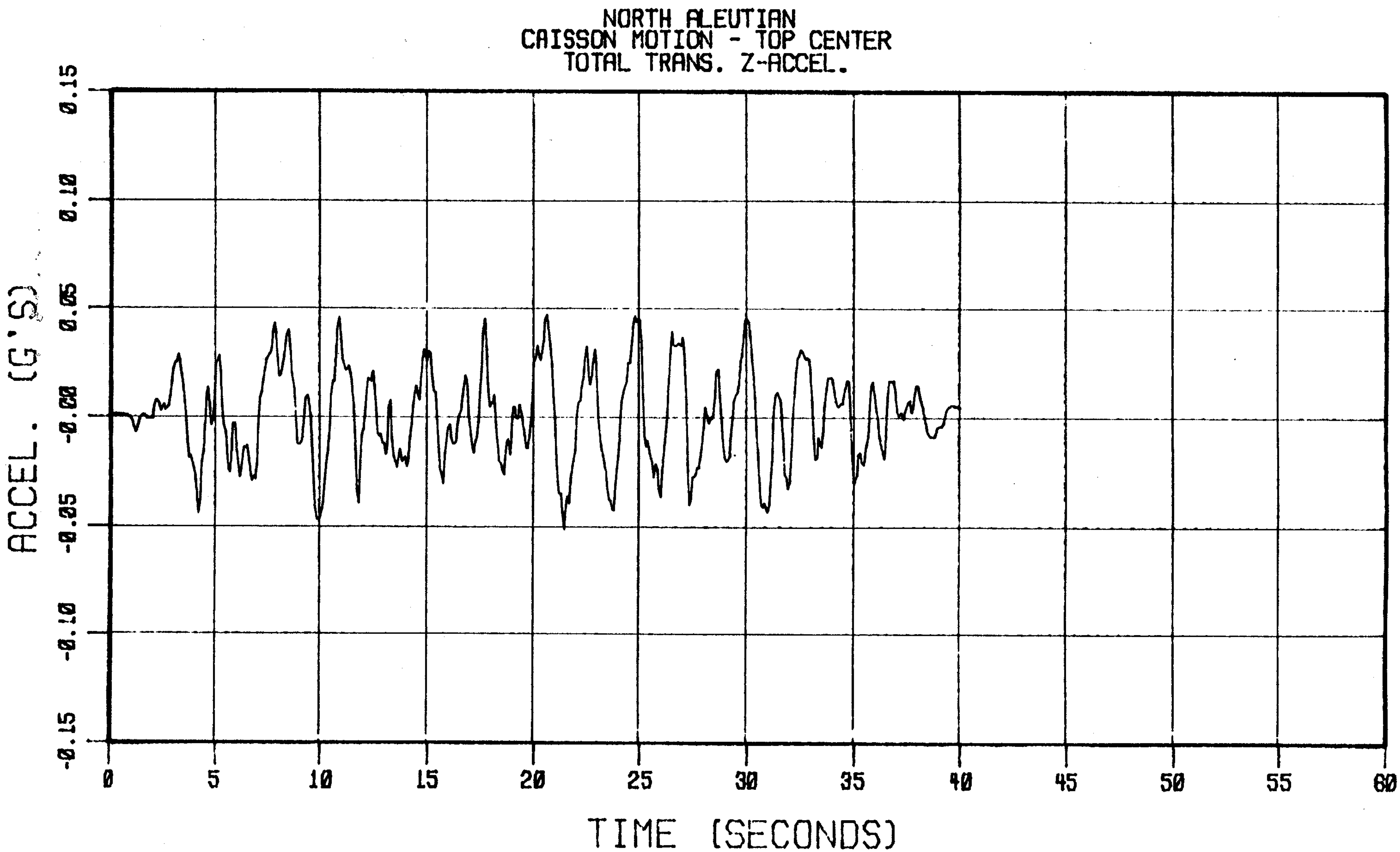


FIGURE 6.2.2 VERTICAL ACCELERATION AT THE CAISSON TOP CENTER

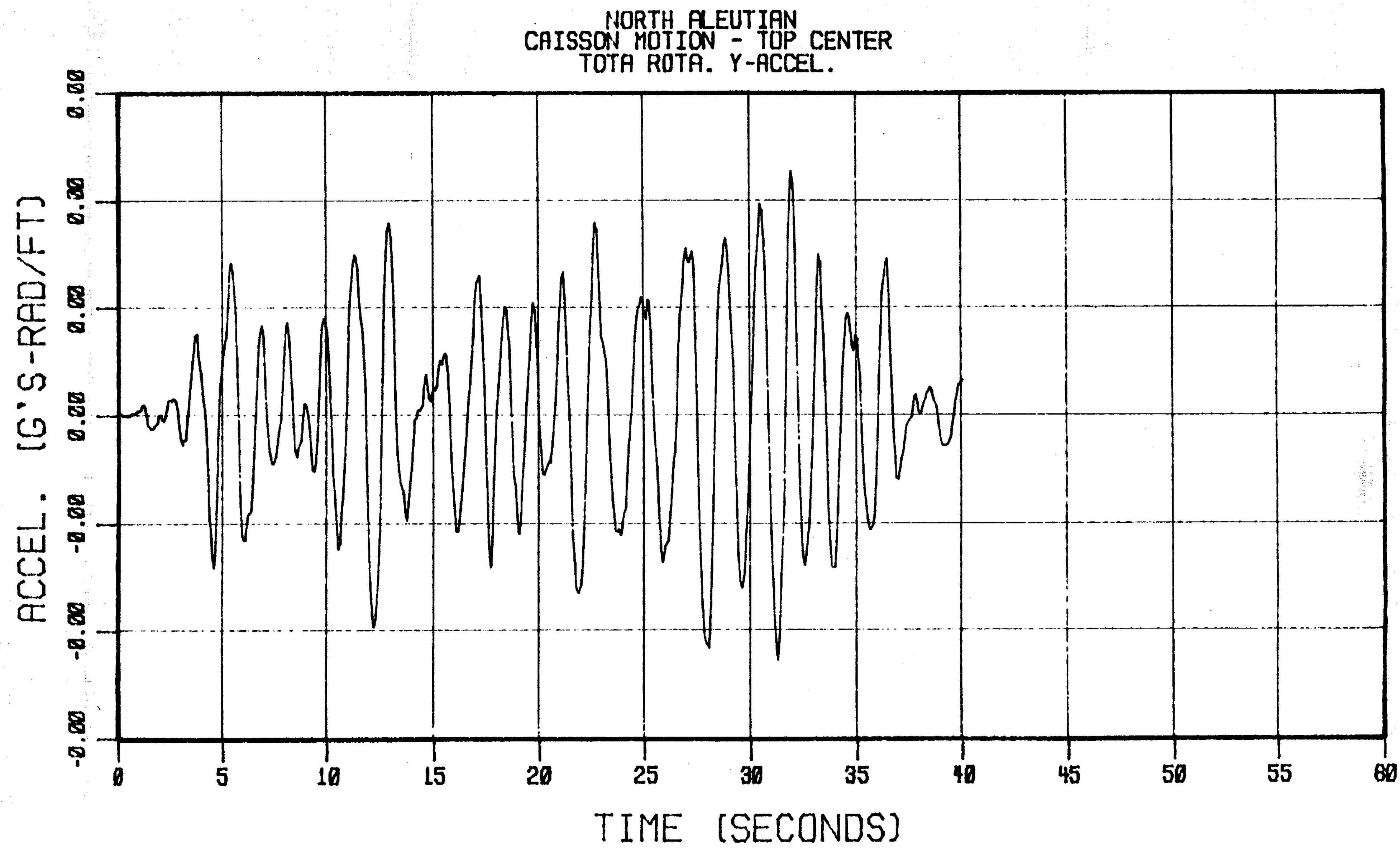


FIGURE 6.2.3 ROTATIONAL ACCELERATION AT THE CAISSON TOP

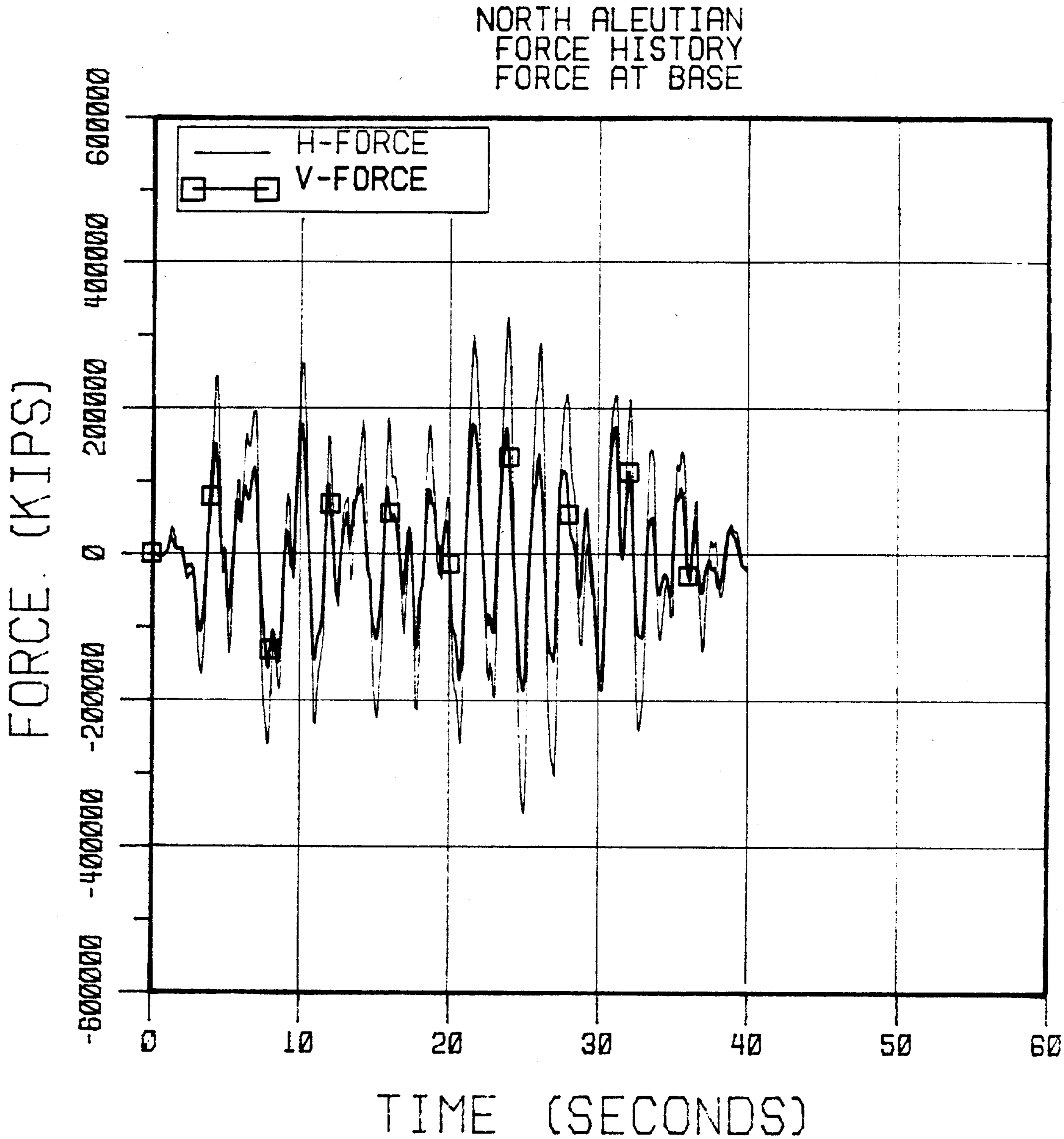


FIGURE 6.2.4 HORIZONTAL (H) AND VERTICAL (V) FORCES ON THE JACKET BASE, 300FT WATER DEPTH

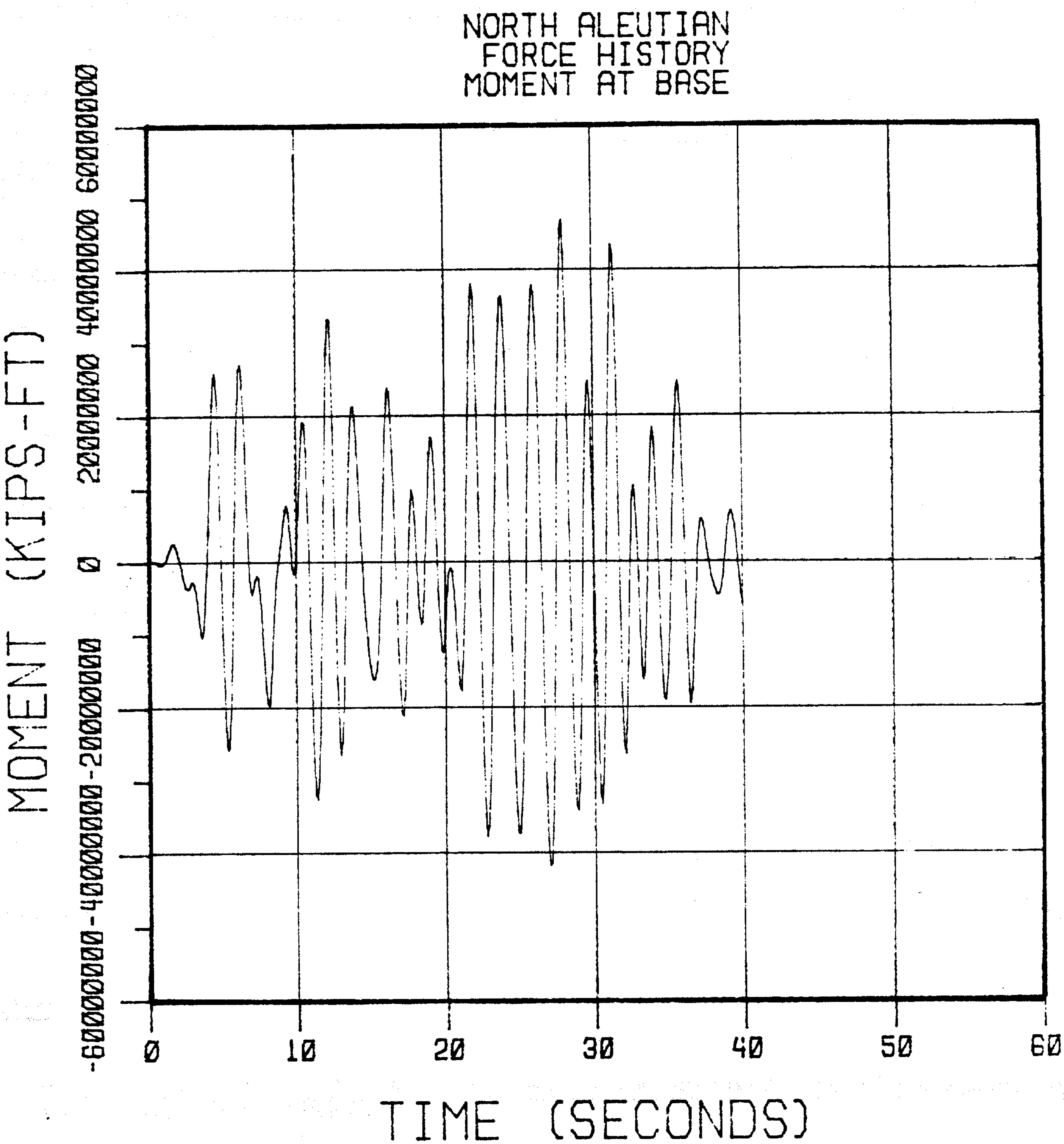


FIGURE 6.2.5 GLOBAL OVERTURNING MOMENT ON THE JACKET BASE, 300 FT WATER DEPTH

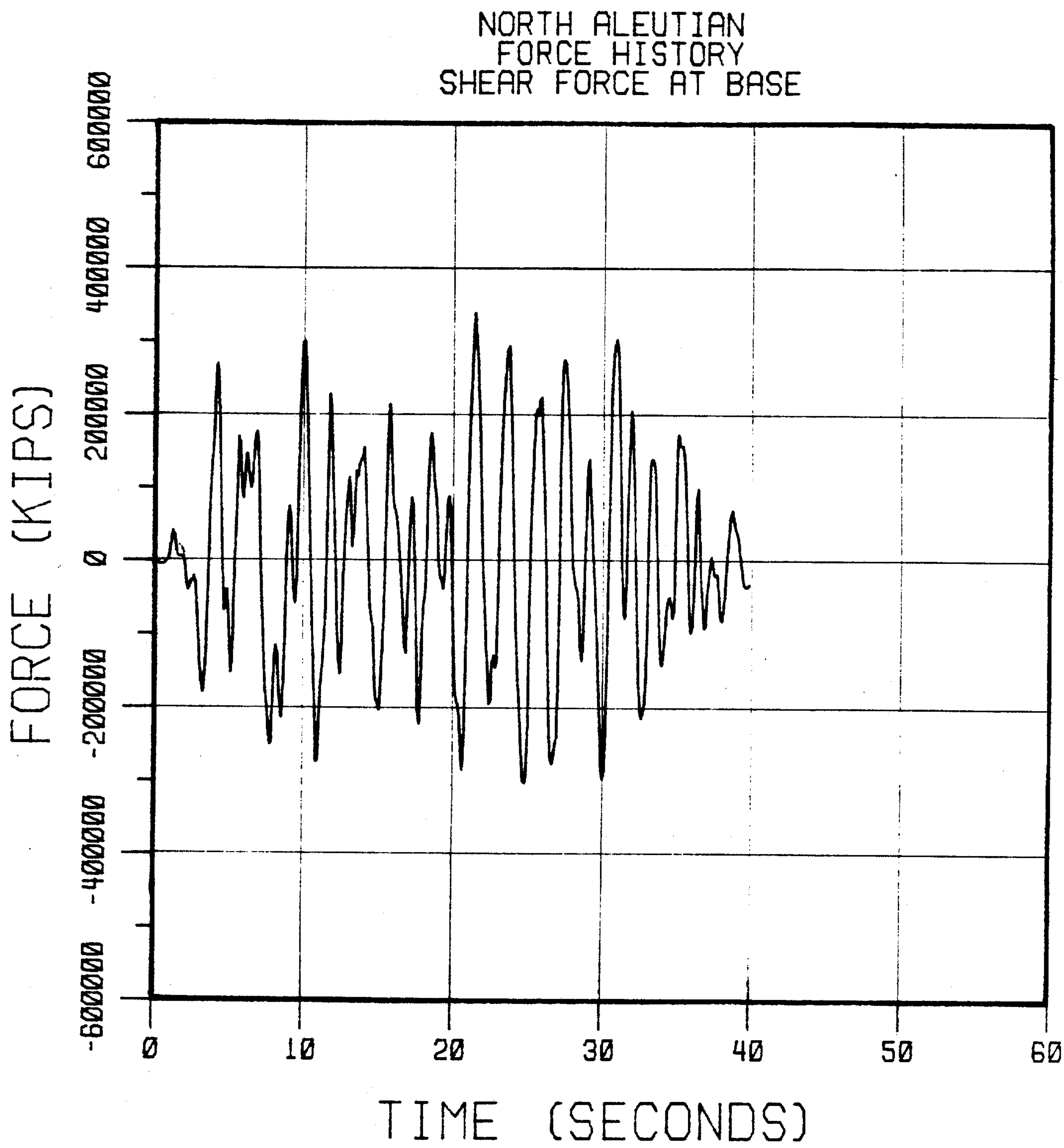


FIGURE 6.2.7 SHEAR FORCE TIME HISTORY AT THE JACKET BASE, 150 FT WATER DEPTH

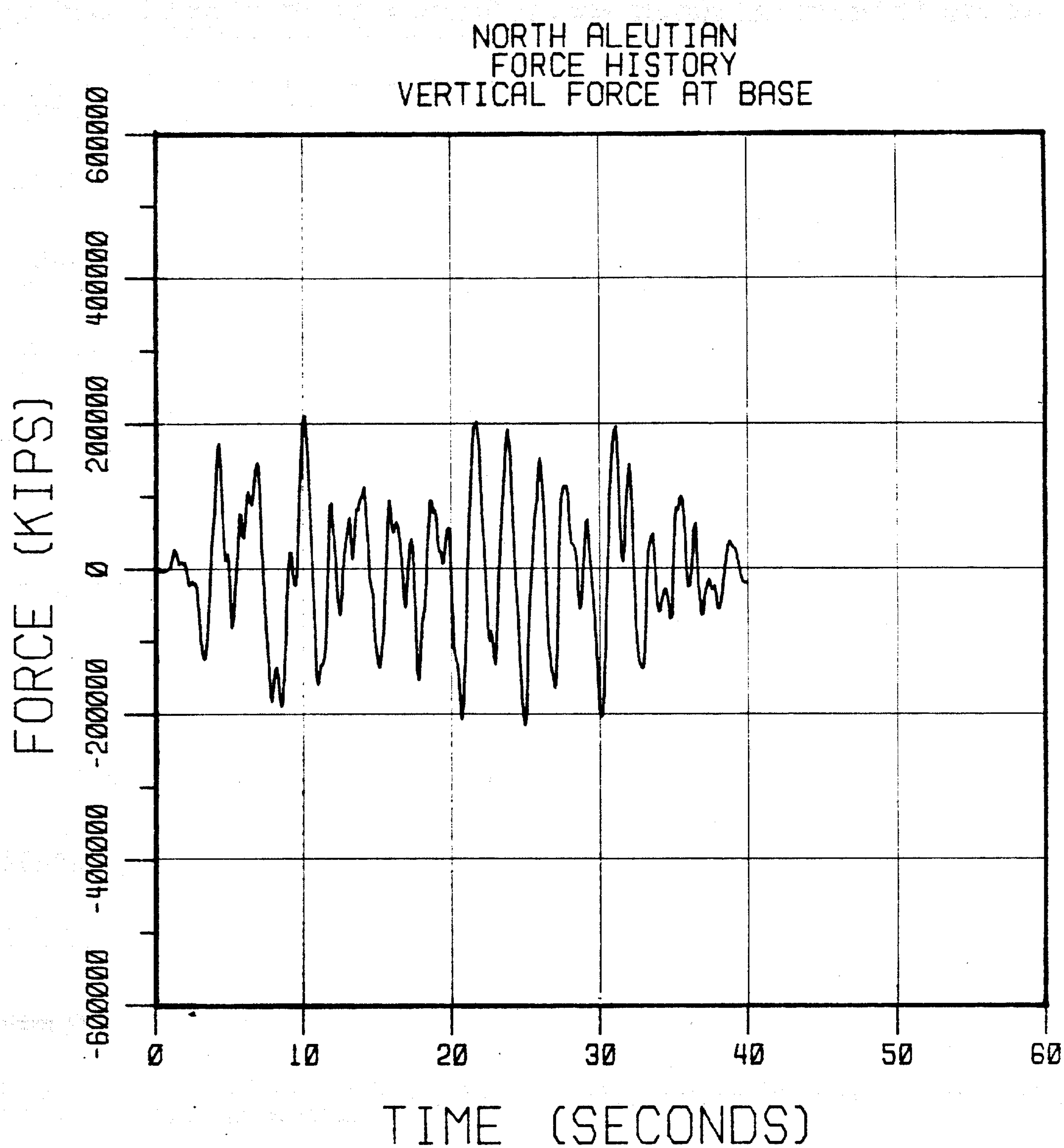


FIGURE 6.2.8 VERTICAL FORCE TIME HISTORY AT THE JACKET BASE, 150 FT WATER DEPTH

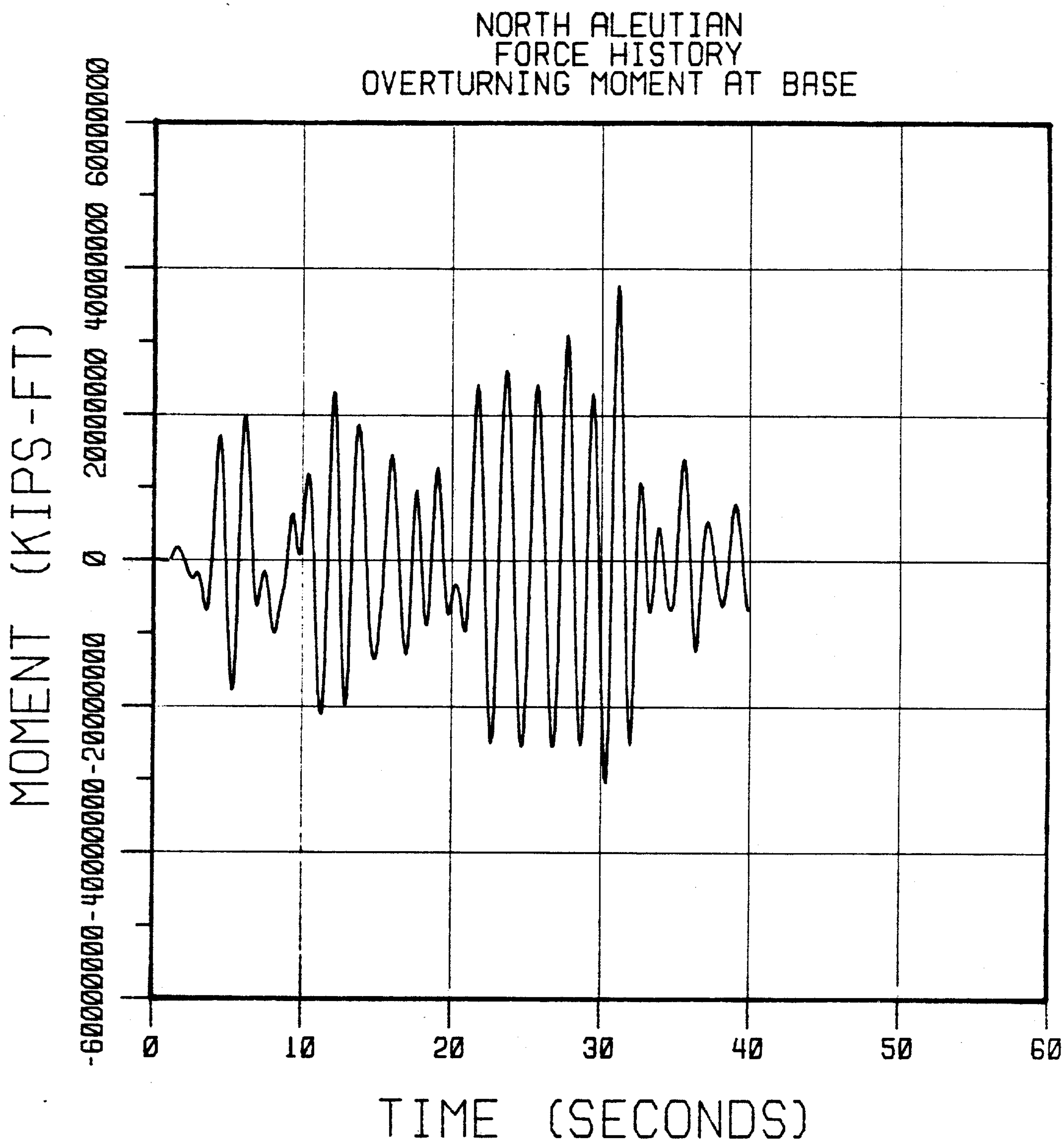


FIGURE 6.2.9 OVERTURNING MOMENT TIME HISTORY AT THE JACKET BASE, 150 FT WATER DEPTH

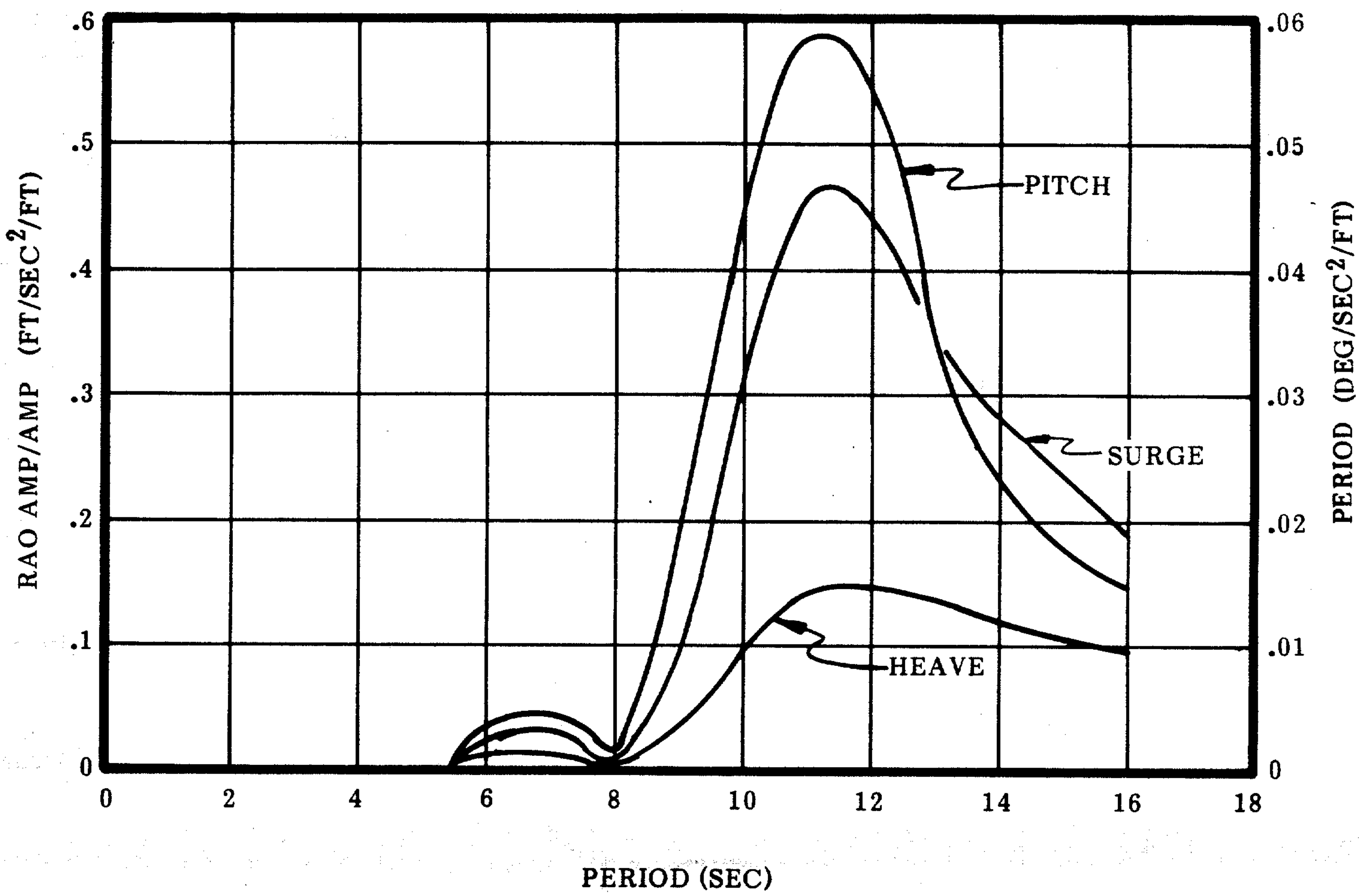


FIGURE 6.2.10 ACCELERATION RAO AT CORNER OF DECK

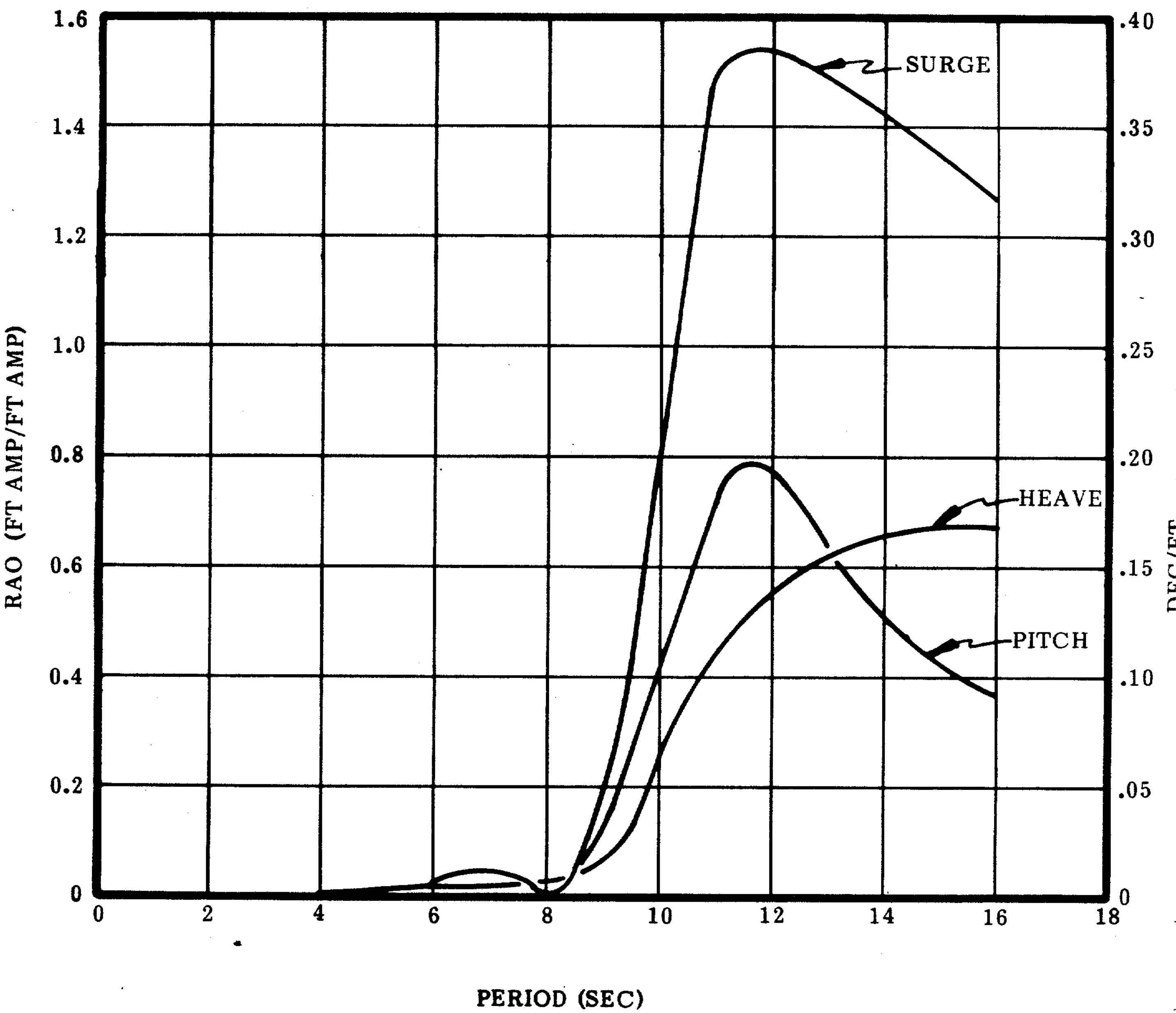


FIGURE 6.2.11 DISPLACEMENT RAO AT CORNER OF DECK

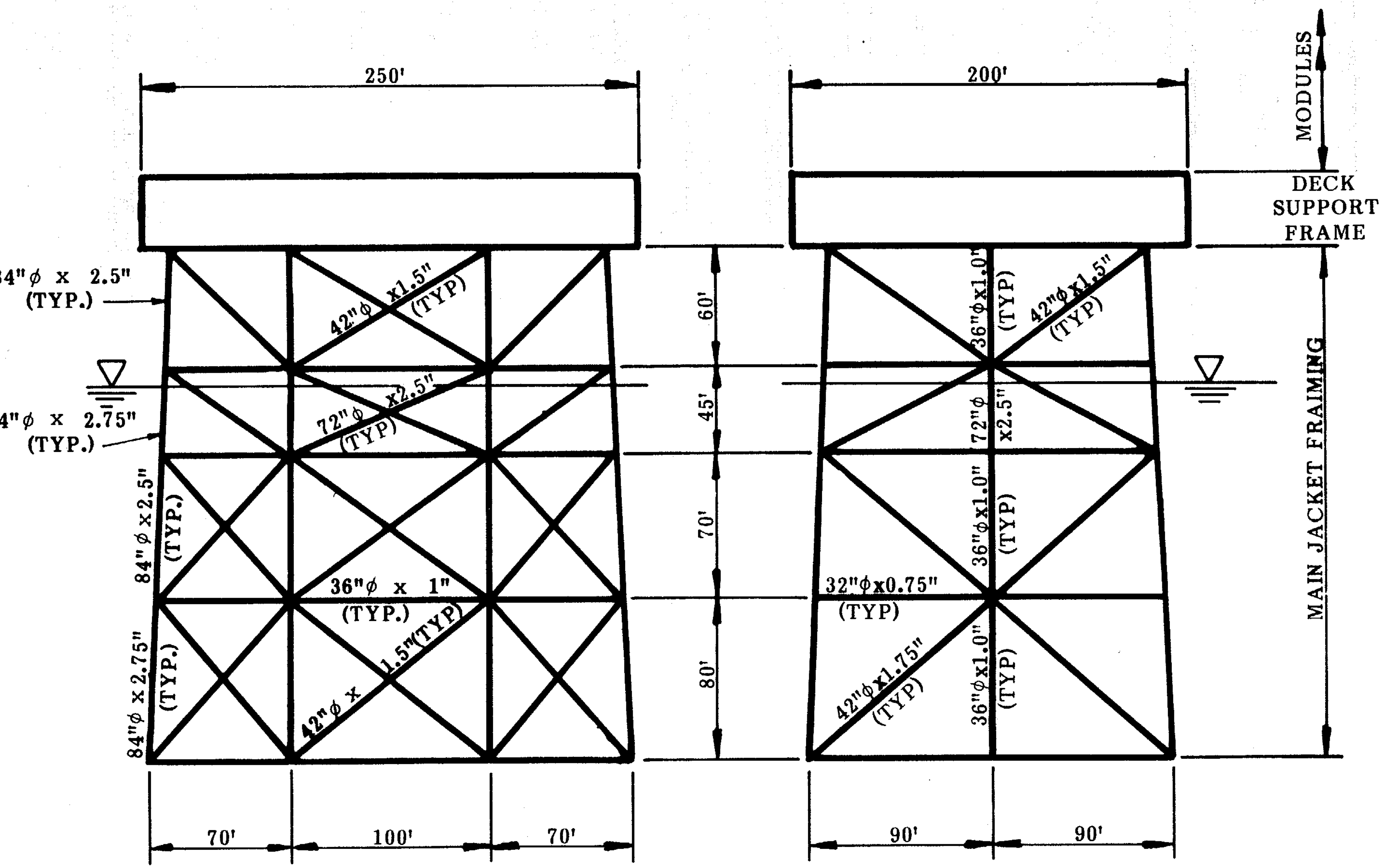


FIGURE 6.2.12 JACKET LAYOUT FOR HYBRID PLATFORM, 300FT WATER DEPTH

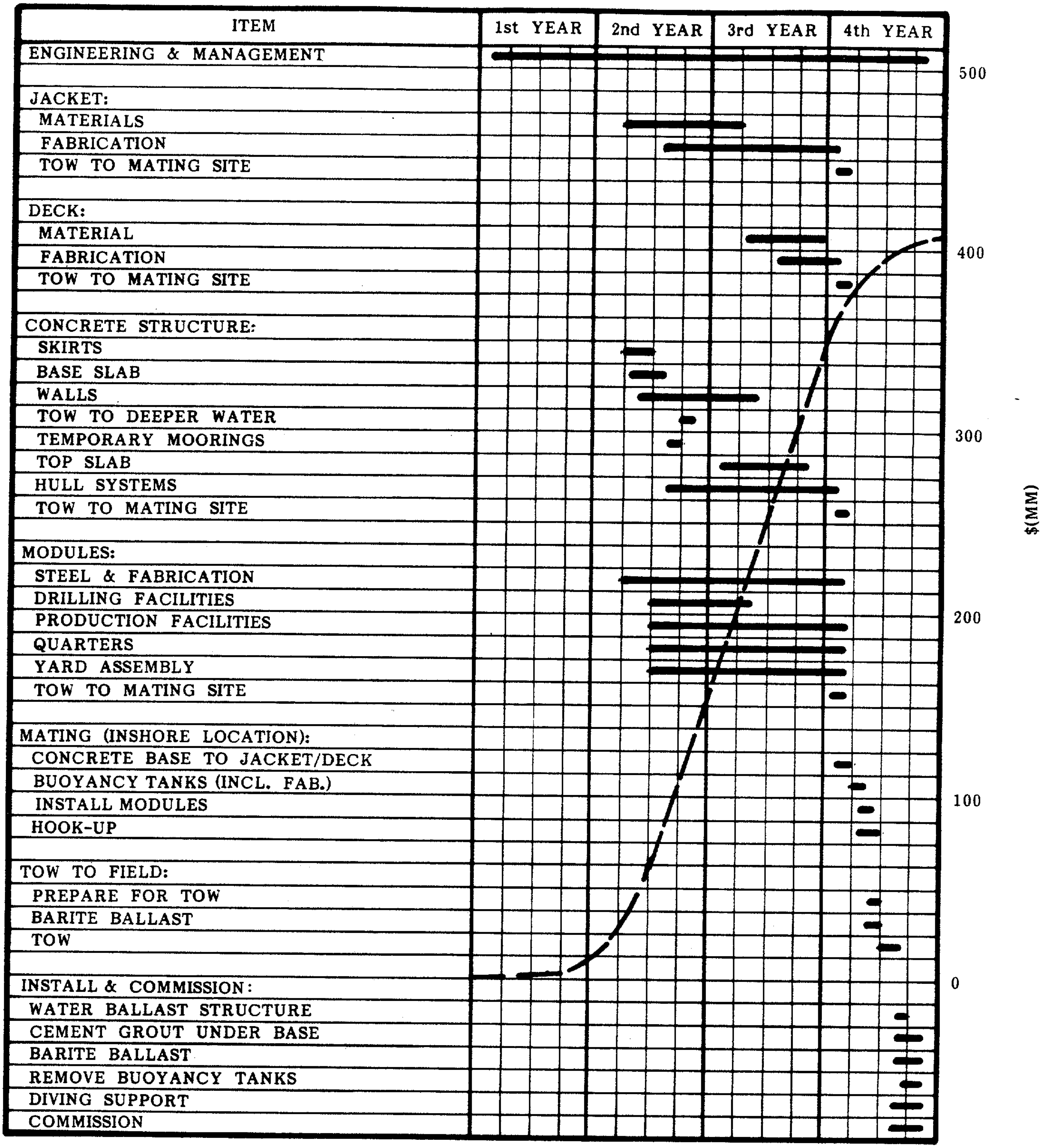
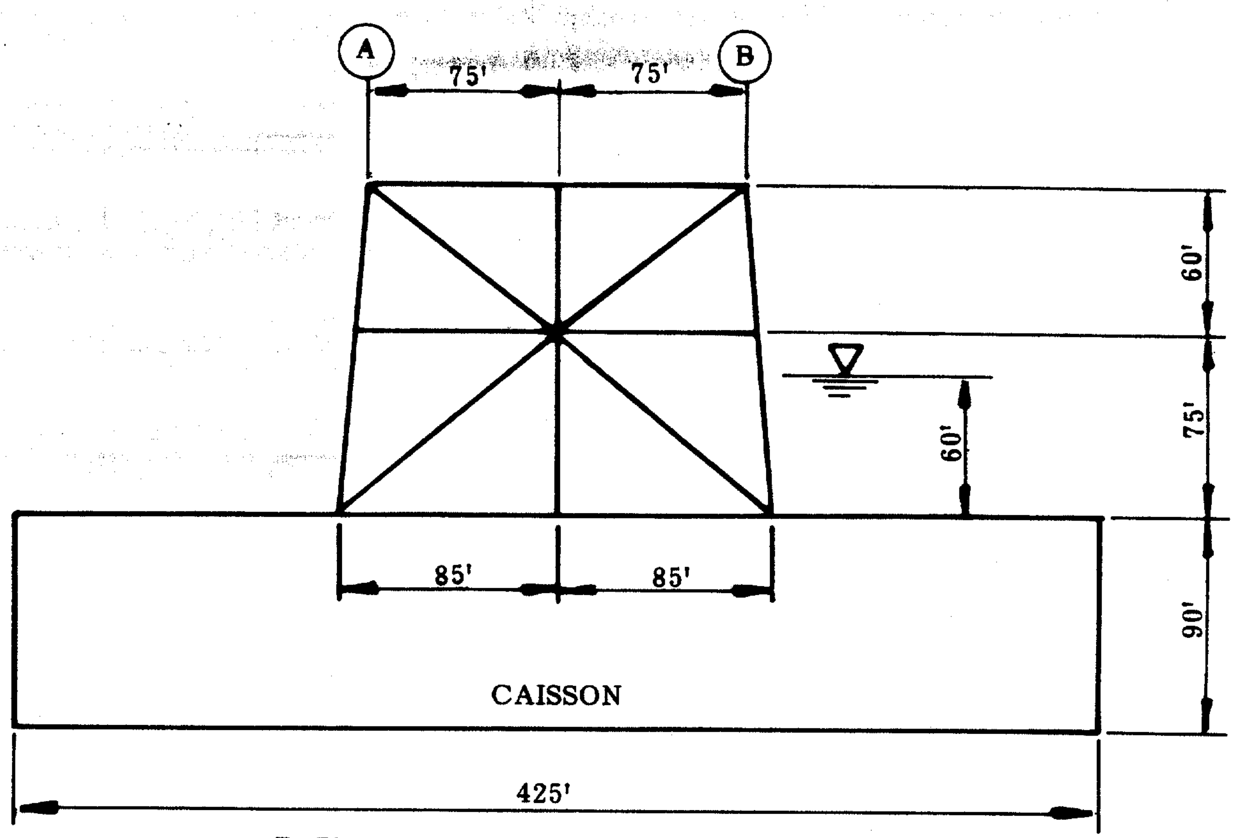
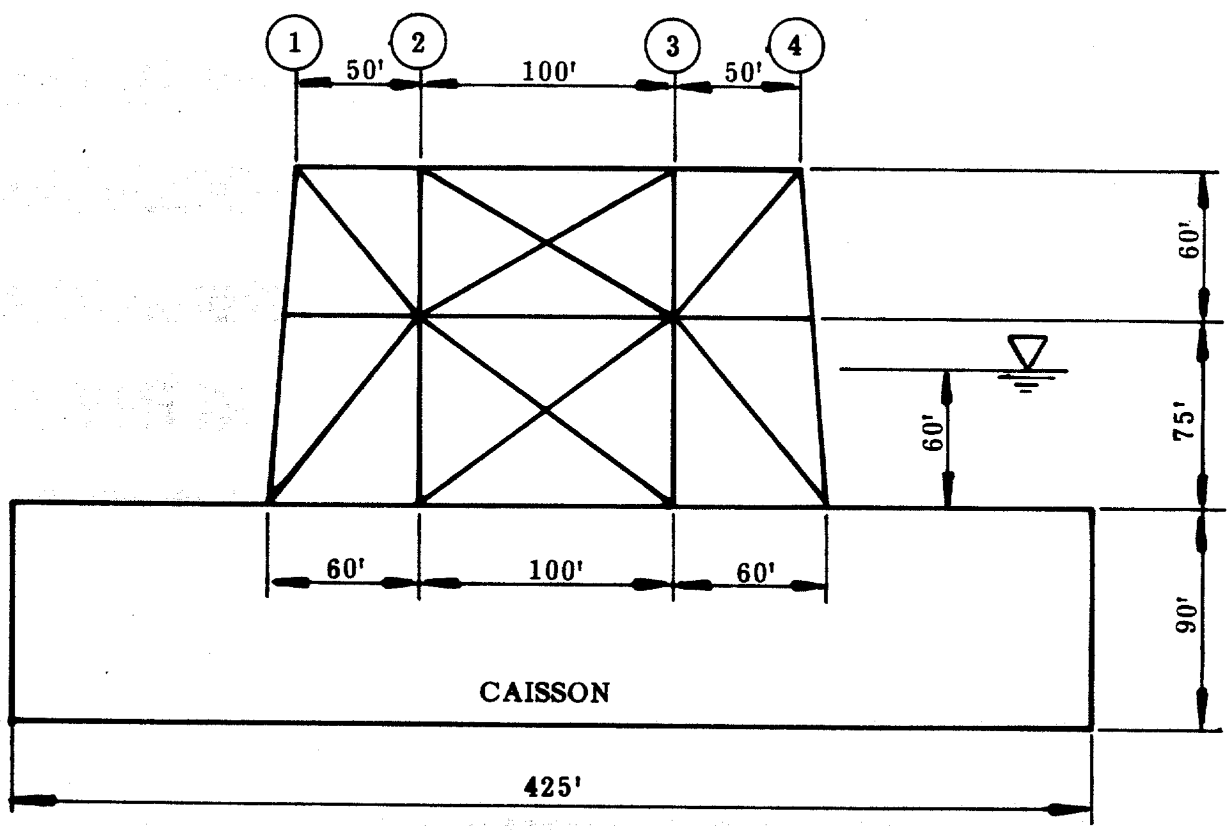


FIGURE 6.3.1 FABRICATION AND INSTALLATION SCHEDULE AND CUMULATIVE CASH FLOW FOR THE HYBRID STRUCTURE IN A 300 FT WATER DEPTH FOR 100,000 BOPD, FABRICATION IN JAPAN



ELEVATION @ ROWS 1, 2, 3, AND 4



ELEVATION AT ROWS A & B

FIGURE 6.4.1 HYBRID PLATFORM IN 150 FT WATER DEPTH

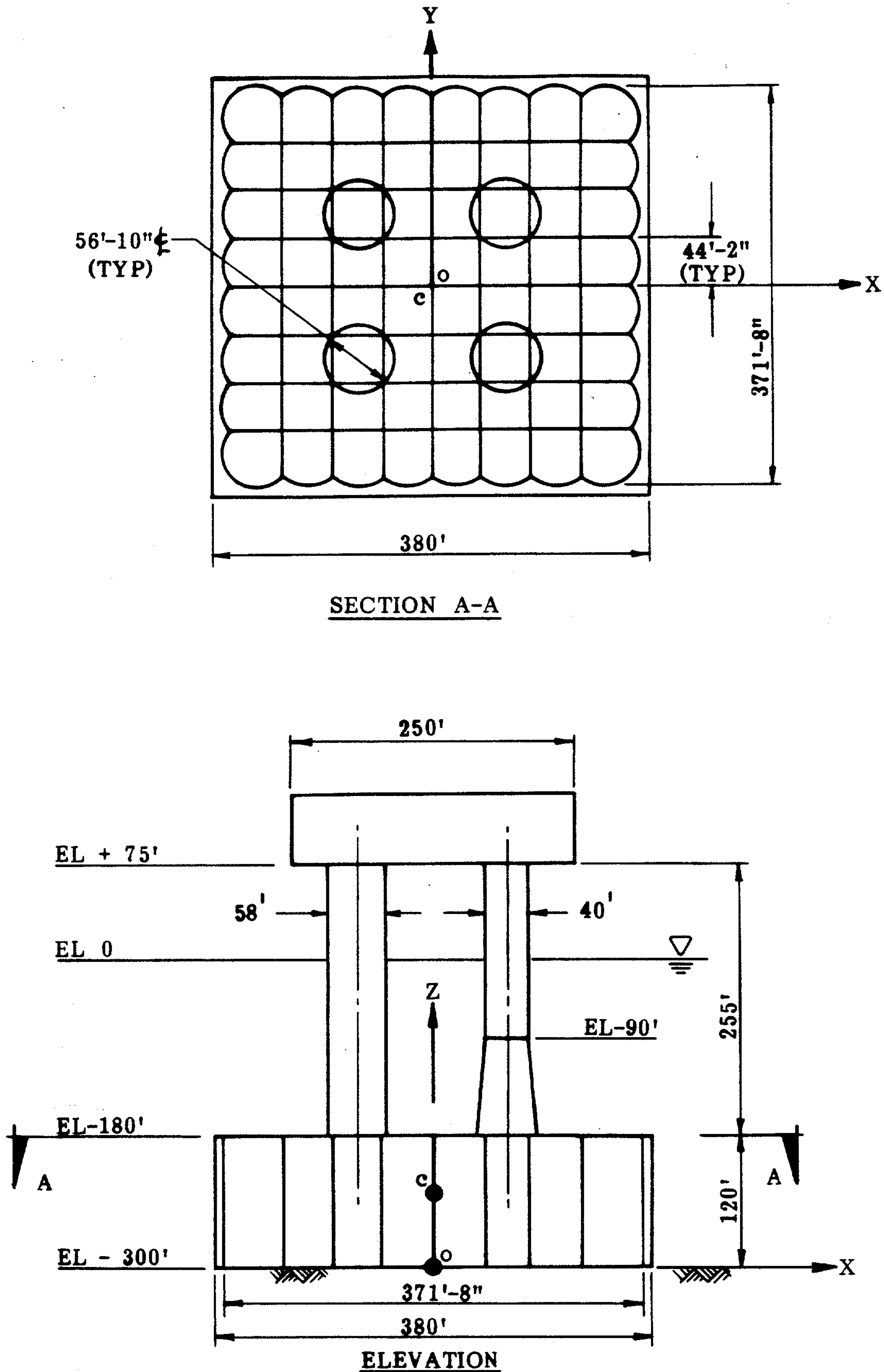


FIGURE 7.1.1 CONCRETE GRAVITY PLATFORM LAYOUT, 300 FT WATER DEPTH

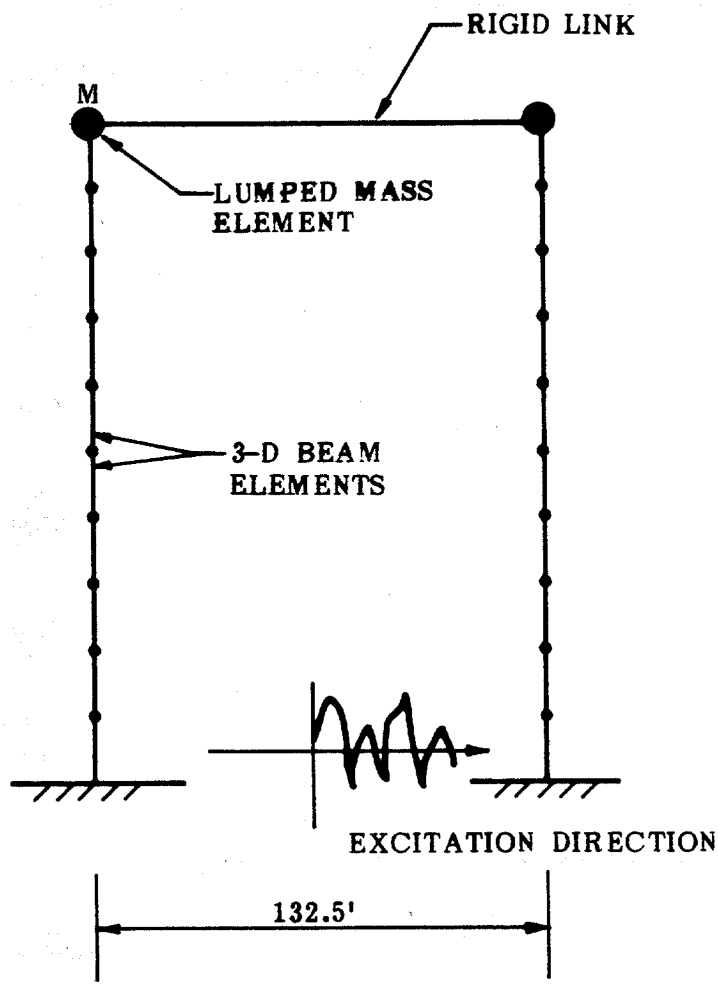
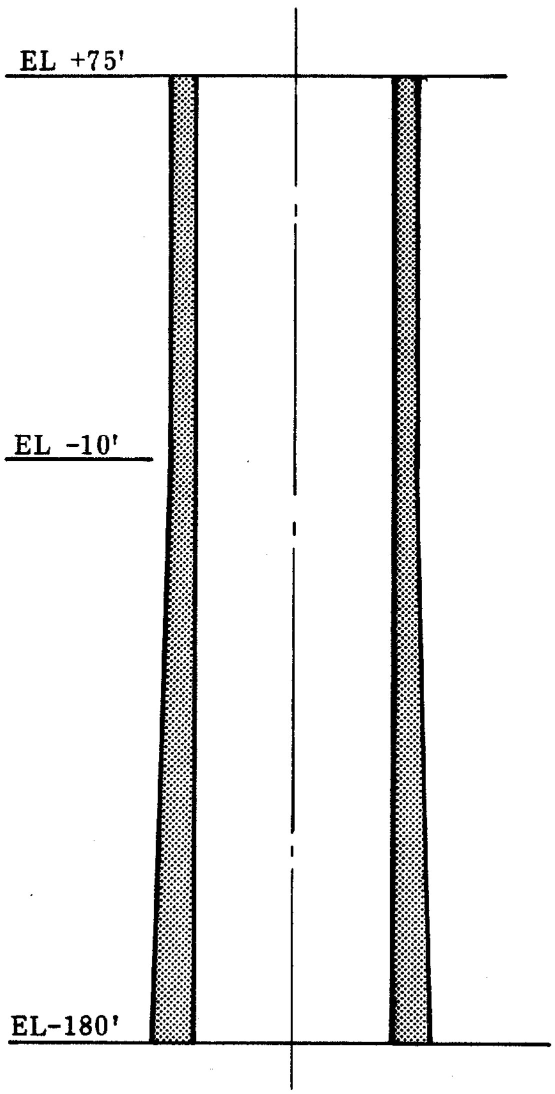


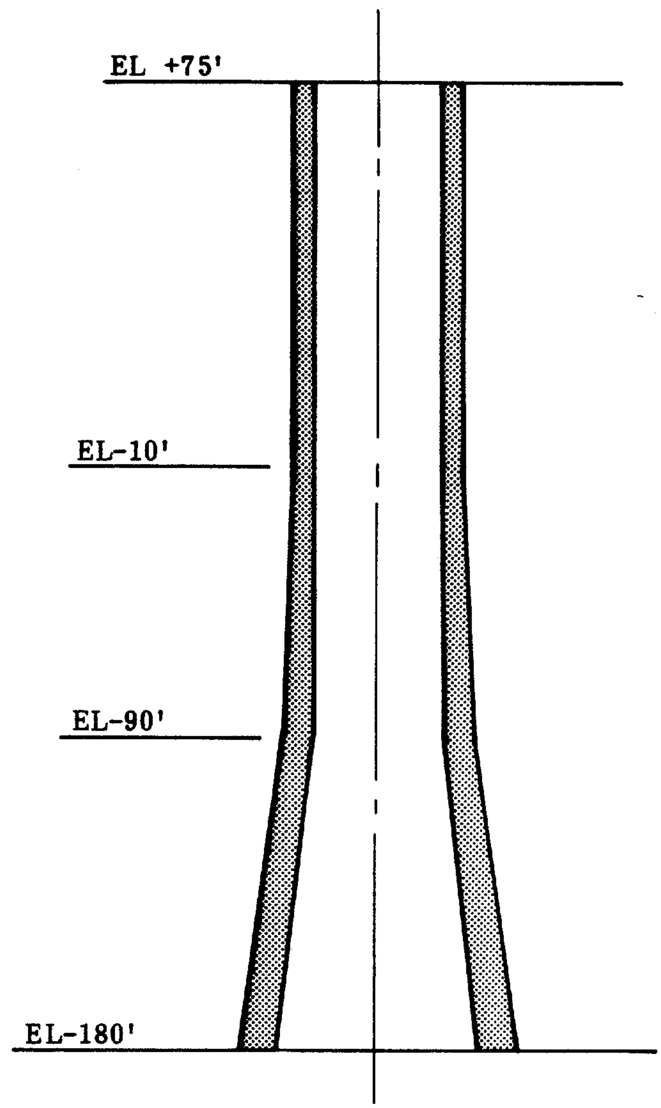
FIGURE 7.1.2 ANALYSIS OF THE TOWER SYSTEM (only 2 towers shown)

PRODUCTION TOWER



ELEV	ID	OD
+75'	54'-0"	58'-0"
-10'	54'-0"	58'-0"
-180'	54'-0"	59'-8"

UTILITY TOWER



ELEV	ID	OD
+75'	36'-0"	40'-0"
-10'	36'-0"	40'-0"
-90'	36'-0"	40'-8"
-180'	54'-0"	59'-8"

FIGURE 7.2.1 SECTIONAL ELEVATION OF TOWERS

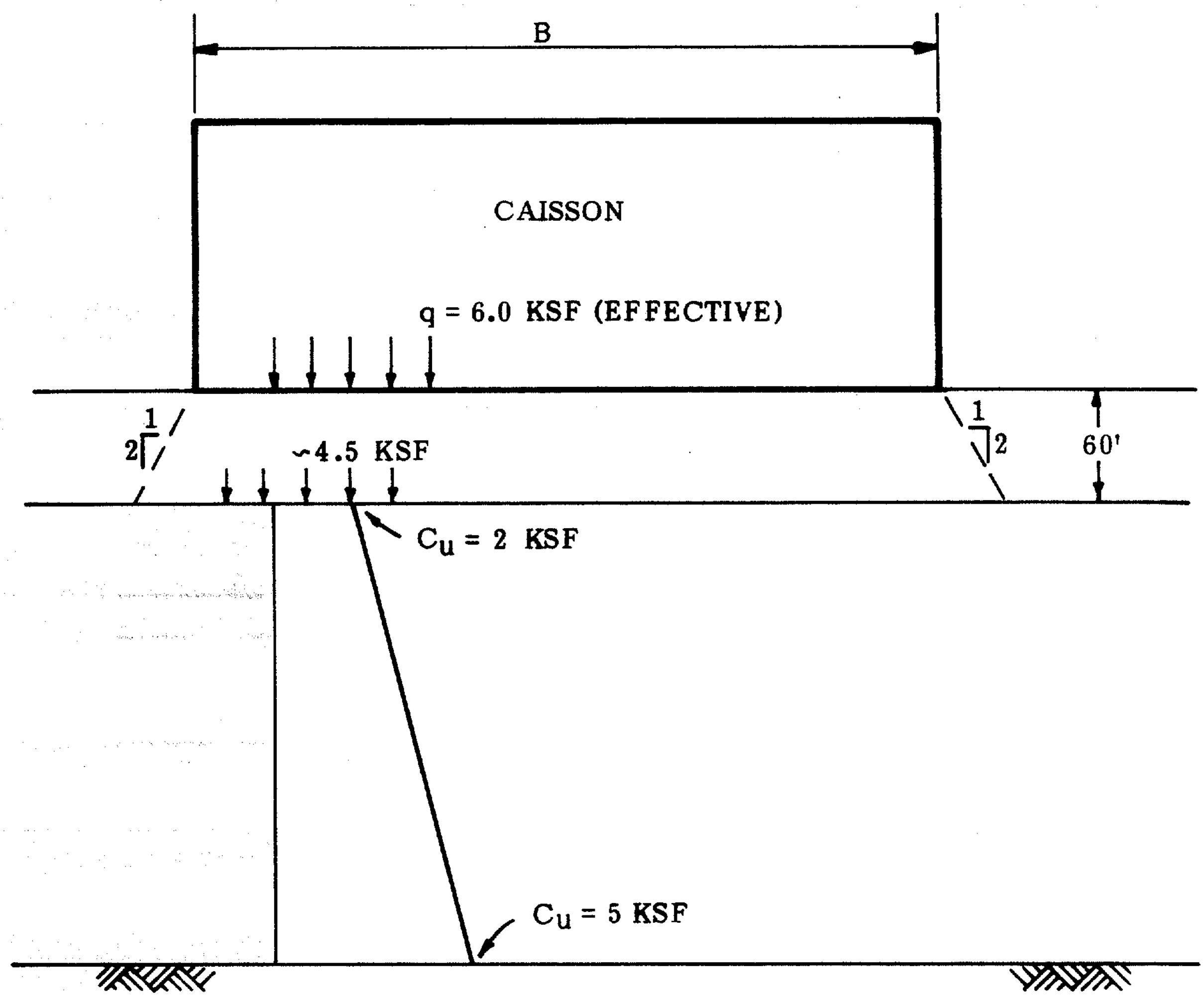


FIGURE 7.3.1 <u>SIMPLIFIED METHOD, BEARING CAPACITY, PROFILE A</u>

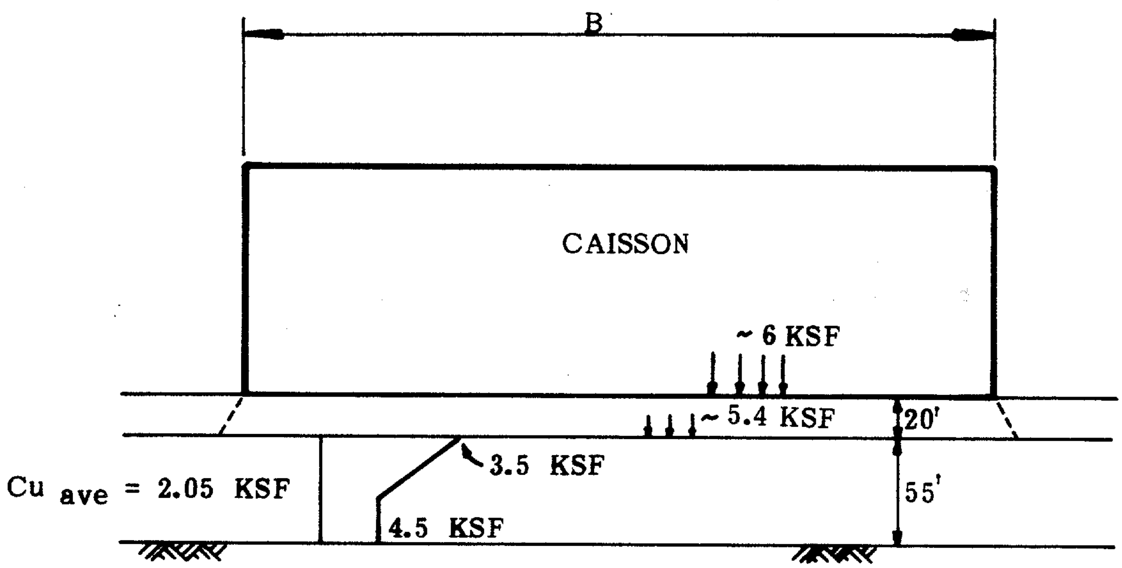


FIGURE 7.3.2 <u>SIMPLIFIED METHOD, BEARING CAPACITY, PROFILE B</u>

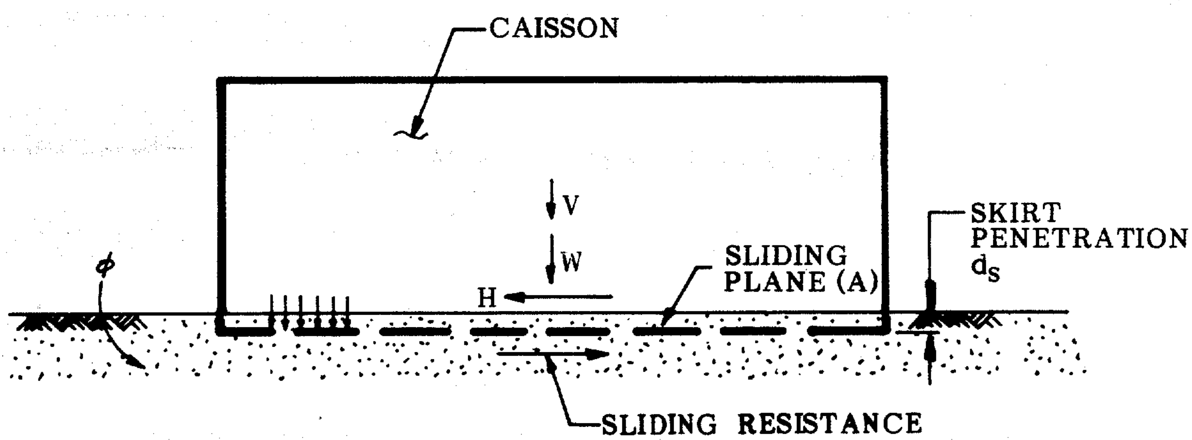


FIGURE 7.3.3 SHALLOW SLIDING STABILITY

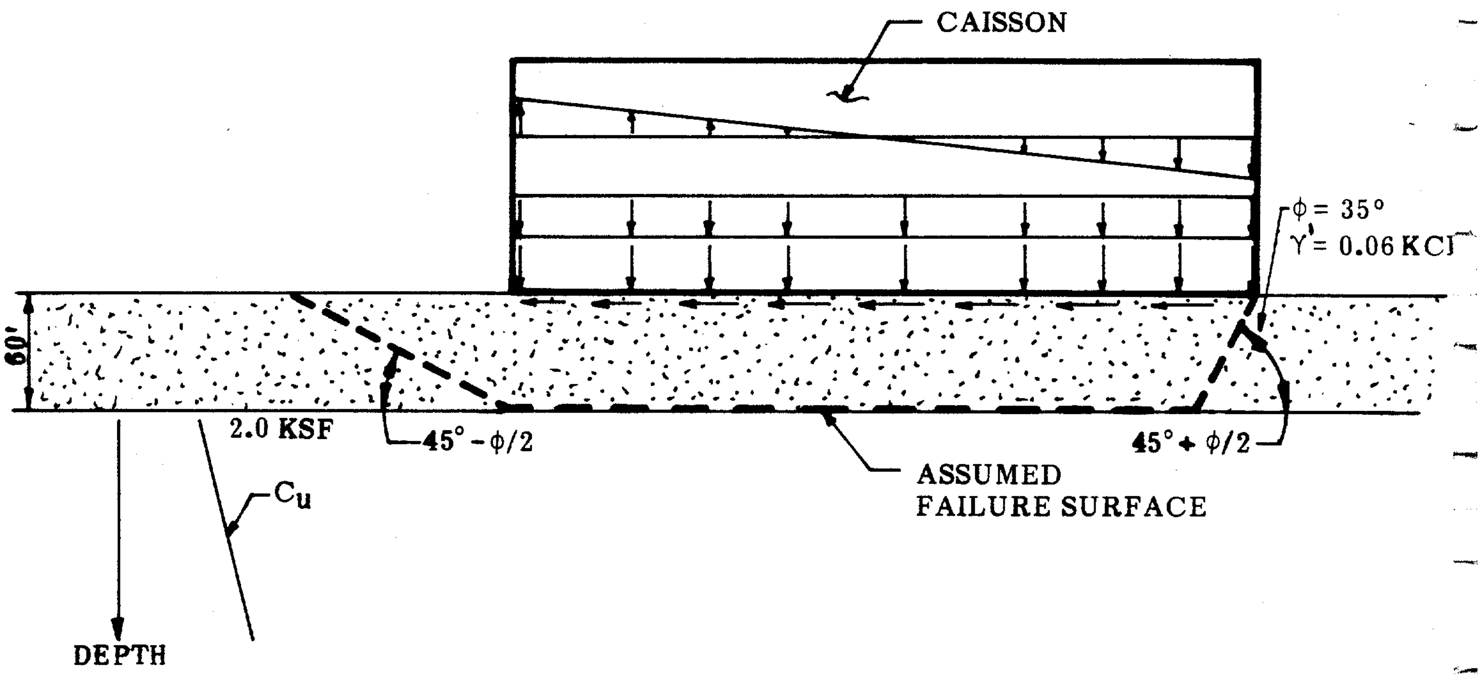


FIGURE 7.3.4 DEEP SLIDING MODE, PROFILE A

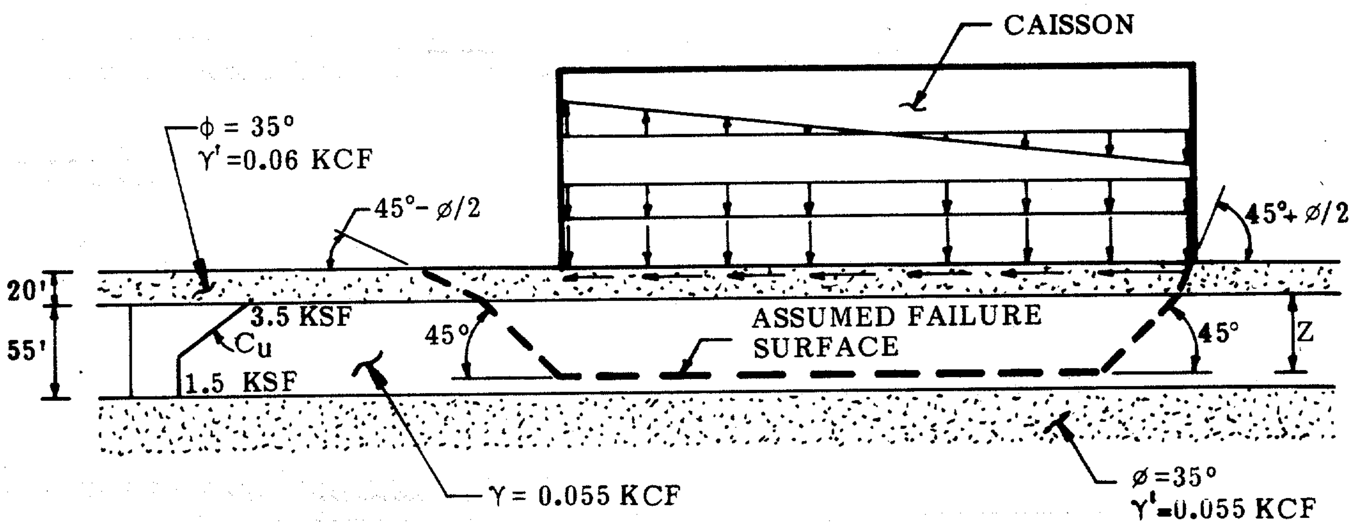


FIGURE 7.3.5 DEEP SLIDING MODE, PROFILE B

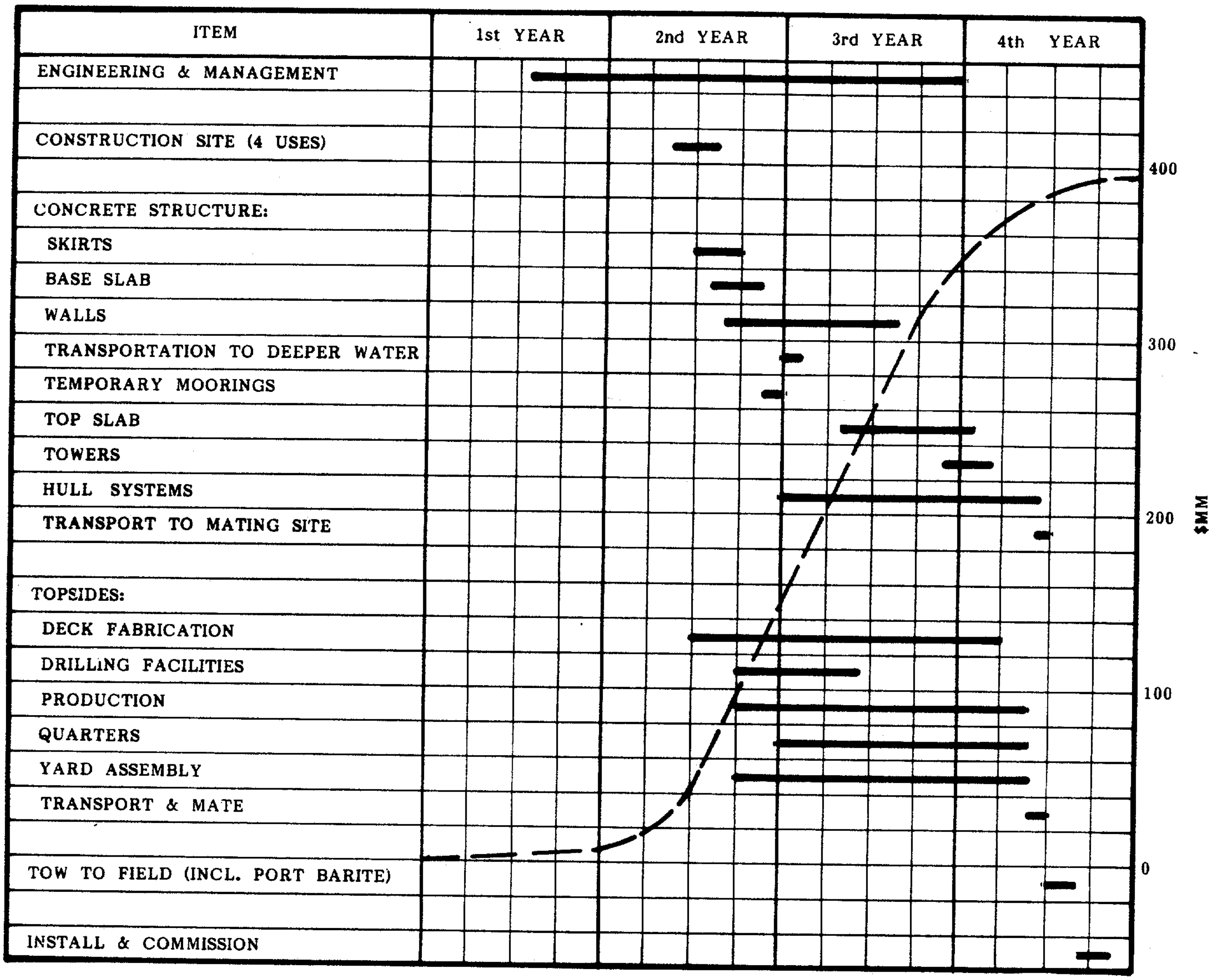


FIGURE 7.4.1 FABRICATION AND INSTALLATION SCHEDULE AND CUMULATIVE CASH FLOW FOR A CONCRETE GRAVITY PLATFORM FABRICATED IN JAPAN FOR 300 FT WATER DEPTH AND 100,000 BOPD

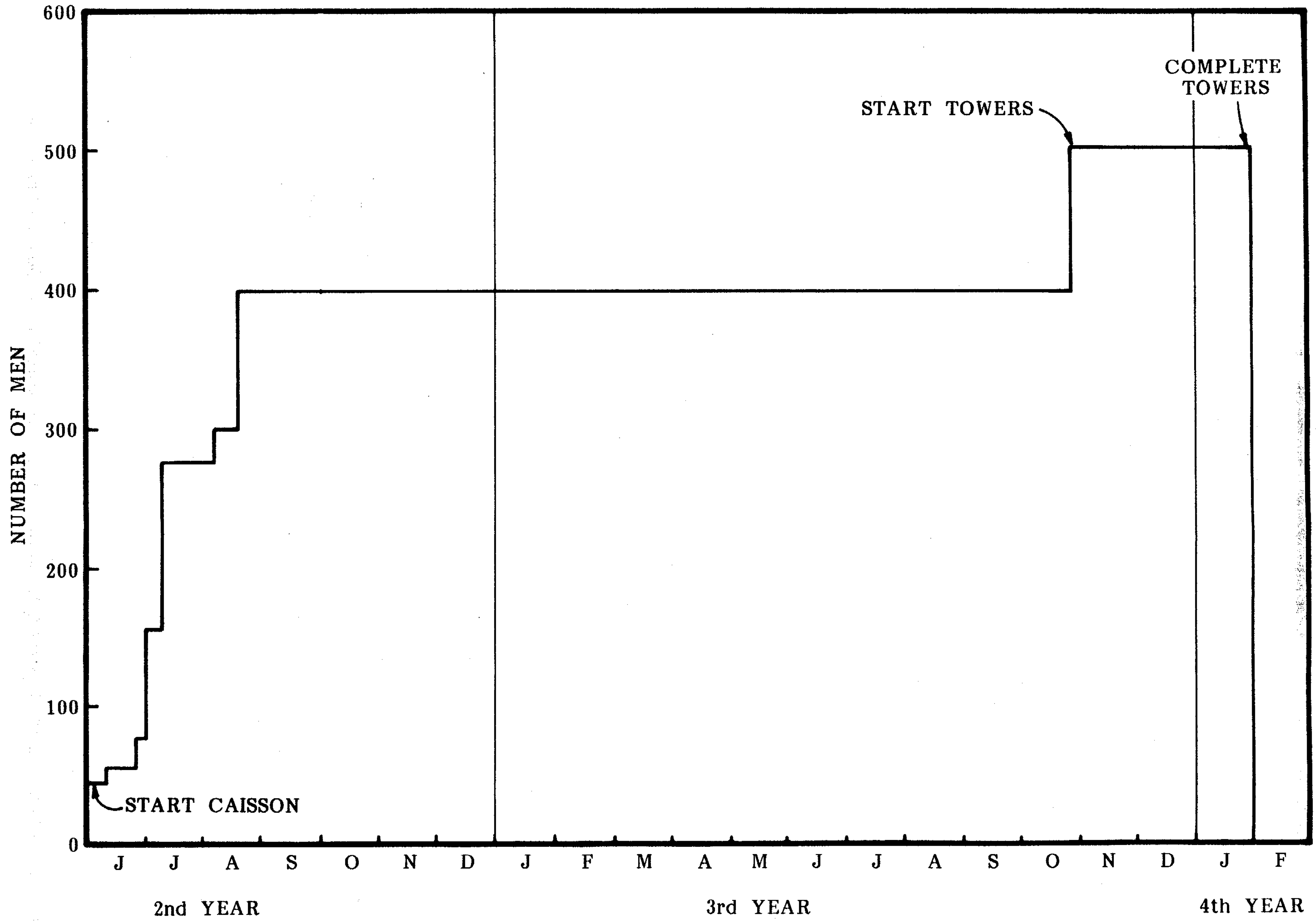


FIGURE 7.4.2 MANPOWER FOR CONCRETE CONSTRUCTION

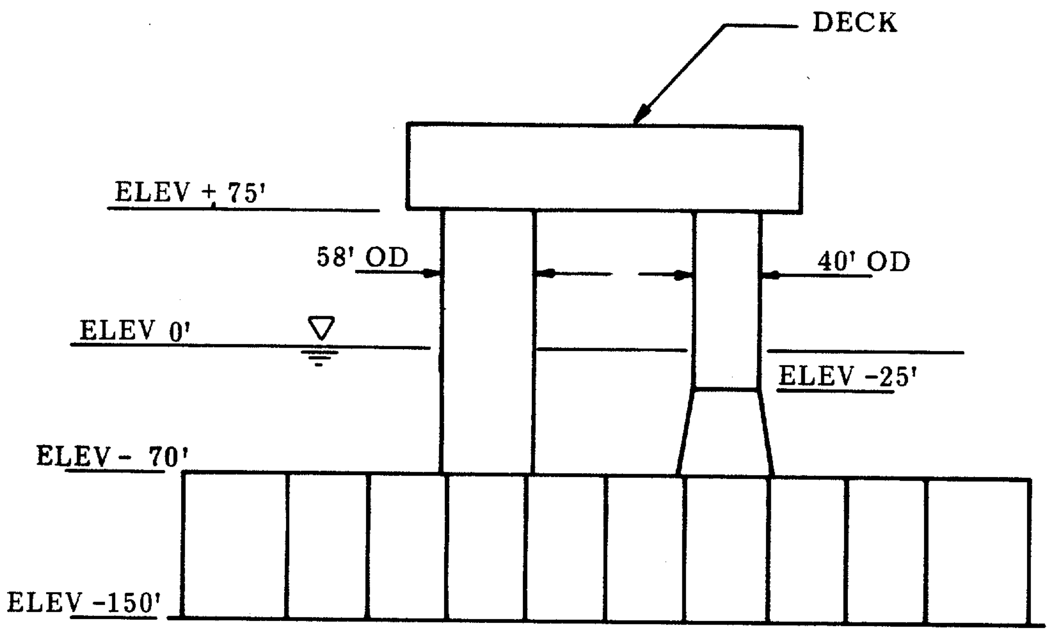


FIGURE 7.5.1 CONCRETE GRAVITY PLATFORM, 150 FT WATER DEPTH

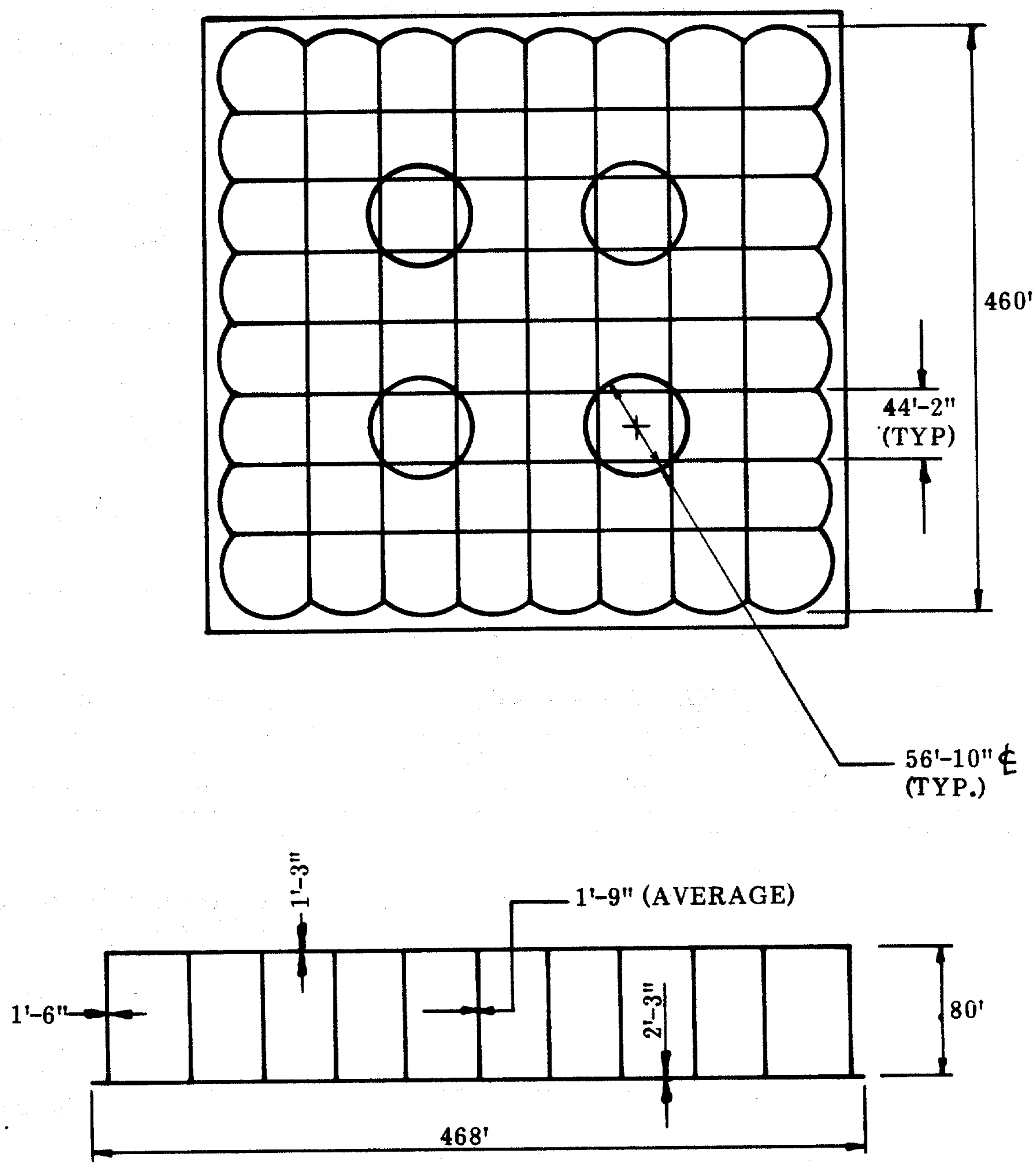


FIGURE 7.5.2 CAISSON DETAILS FOR 150 FT WATER DEPTH CONCRETE GRAVITY PLATFORM

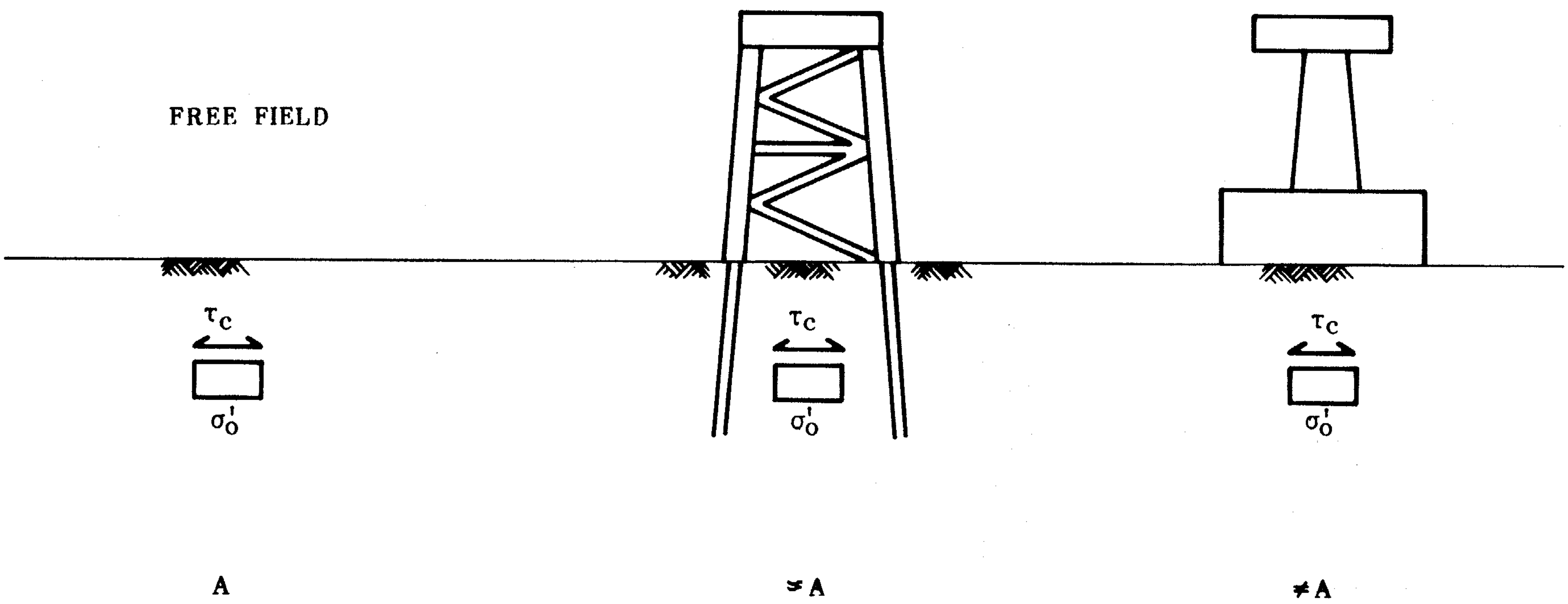
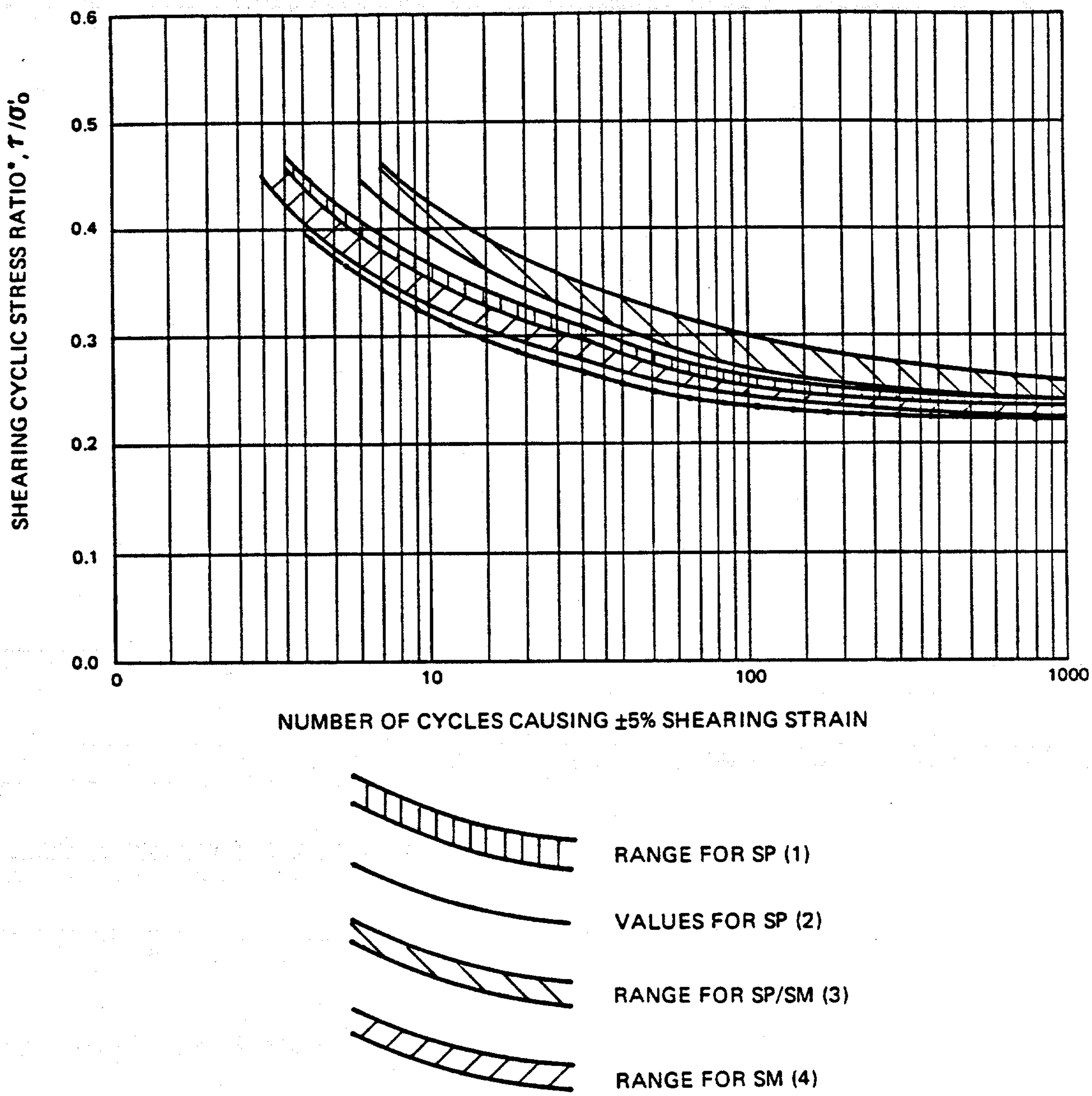
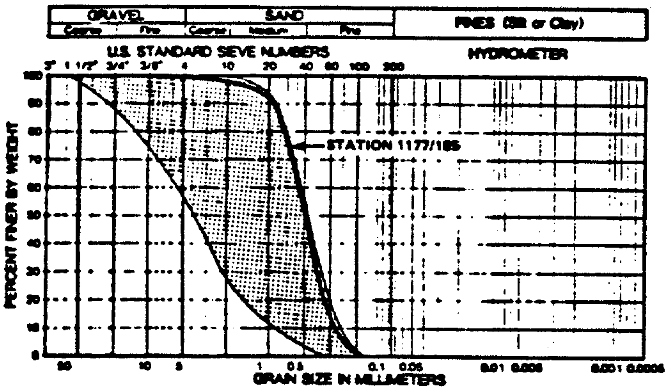


FIGURE 8.1.1 STRUCTURE INFLUENCES ON INITIAL STRESSES AND CYCLIC STRESSES IN FOUNDATION

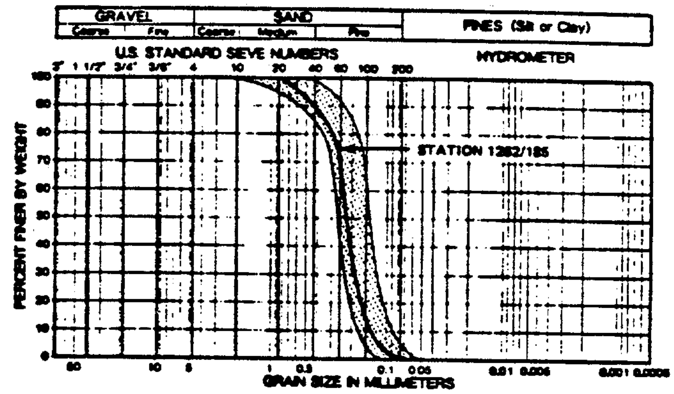


*DEFINED AS RATIO OF CYCLIC SHEARING STRESS (τ) TO INITIAL EFFECTIVE VERTICAL STRESS (σ'_o)

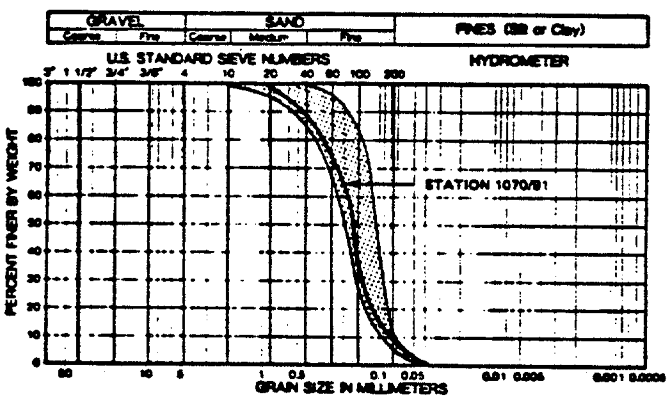
FIGURE 8.1.2 ESTIMATED LIQUEFACTION STRENGTH IN THE FIELD



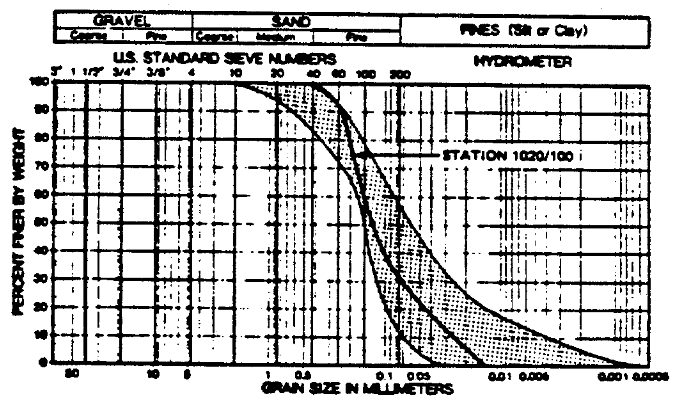
SOIL TYPE NO. 1–SP (1)



SOIL TYPE NO. 2–SP (2)



SOIL TYPE NO. 3–SP/SM (3)



SOIL TYPE NO. 4–SM (4)

FIGURE 8.1.3 RANGE OF GRAIN-SIZE CONDITIONS

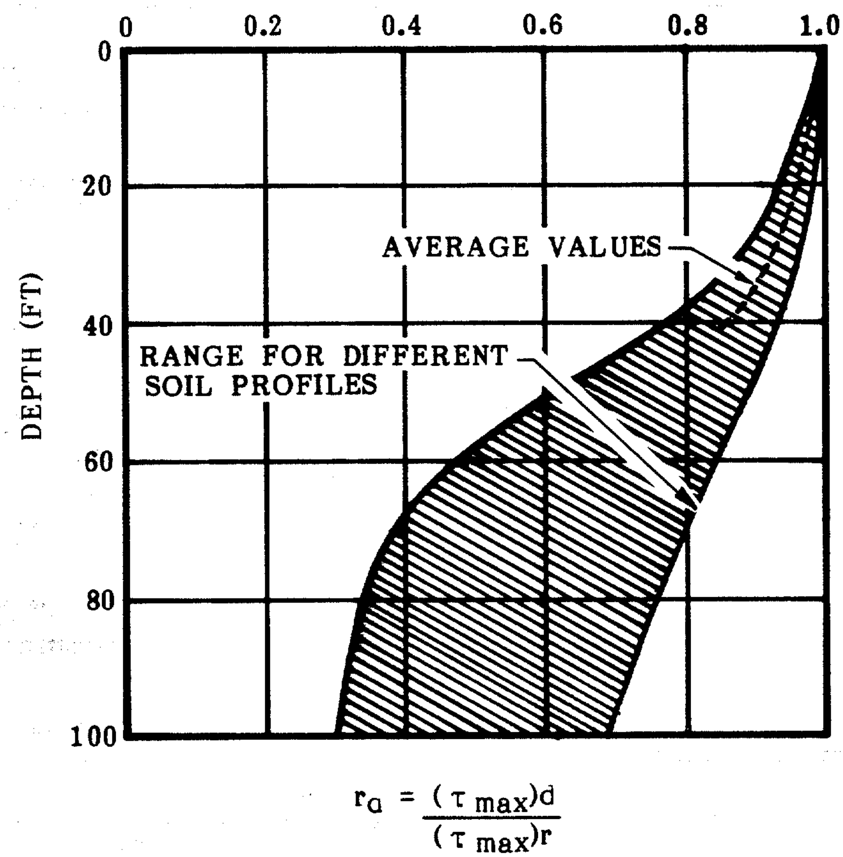


FIGURE 8.2.1 <u>RANGE OF VALUES OF r/d FOR DIFFERENT SOIL PROFILES</u>

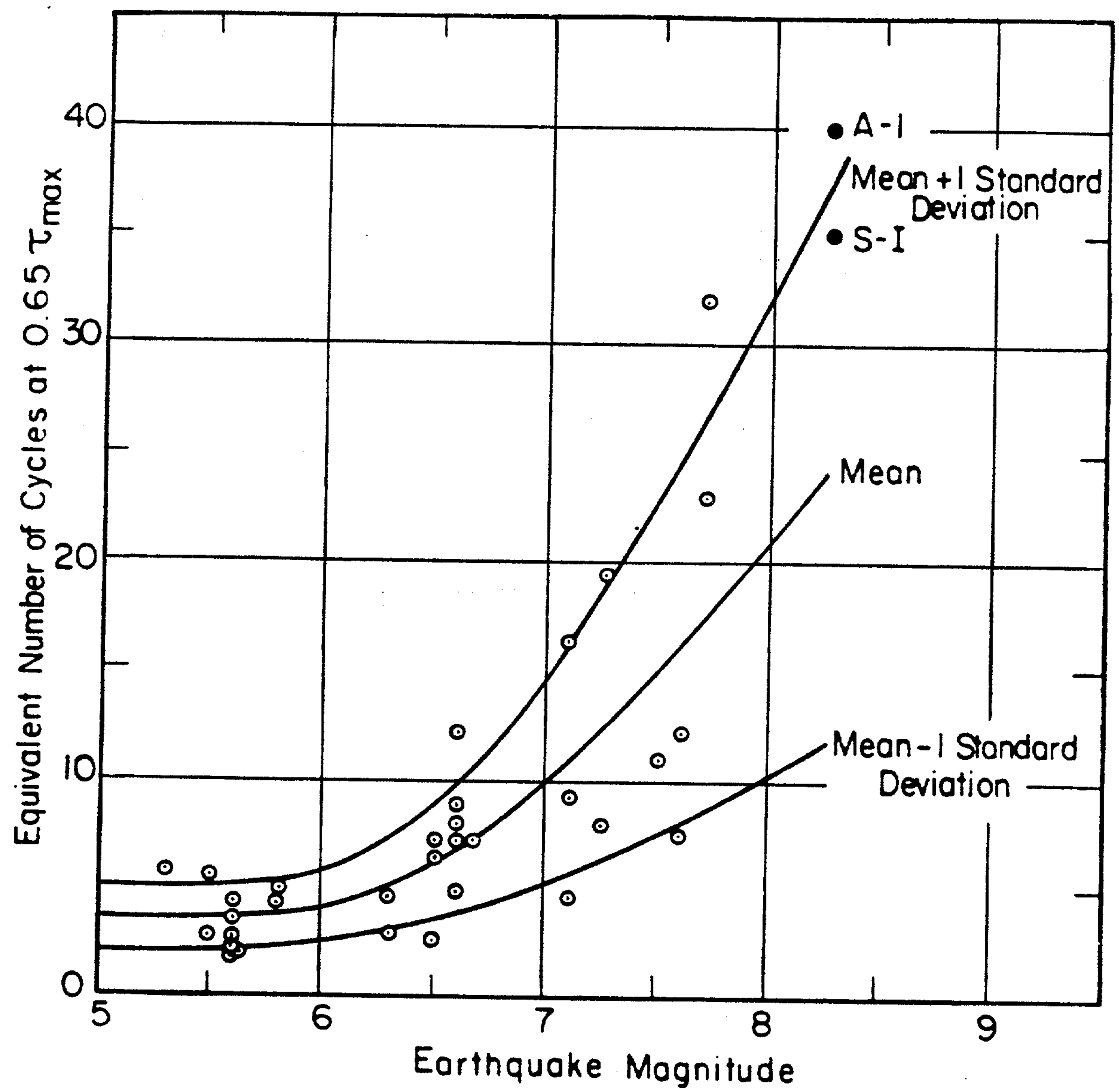


FIGURE 8.2.2 EQUIVALENT NUMBERS OF UNIFORM STRESS CYCLES BASED ON STRONGEST COMPONENTS OF GROUND MOTION

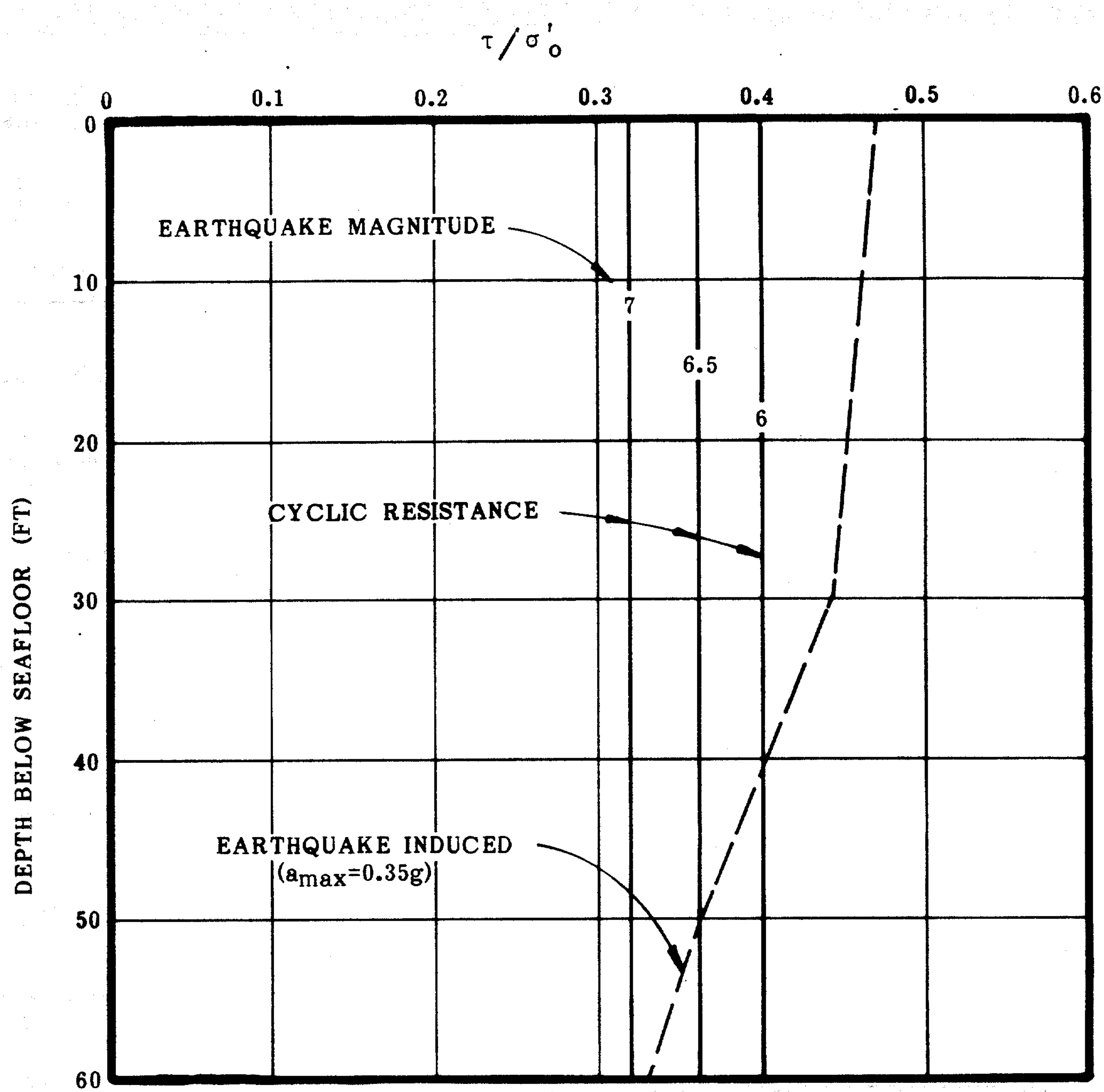


FIGURE 8.2.3 RESULTS FOR FREE-FIELD CONDITIONS, LOWER BOUND

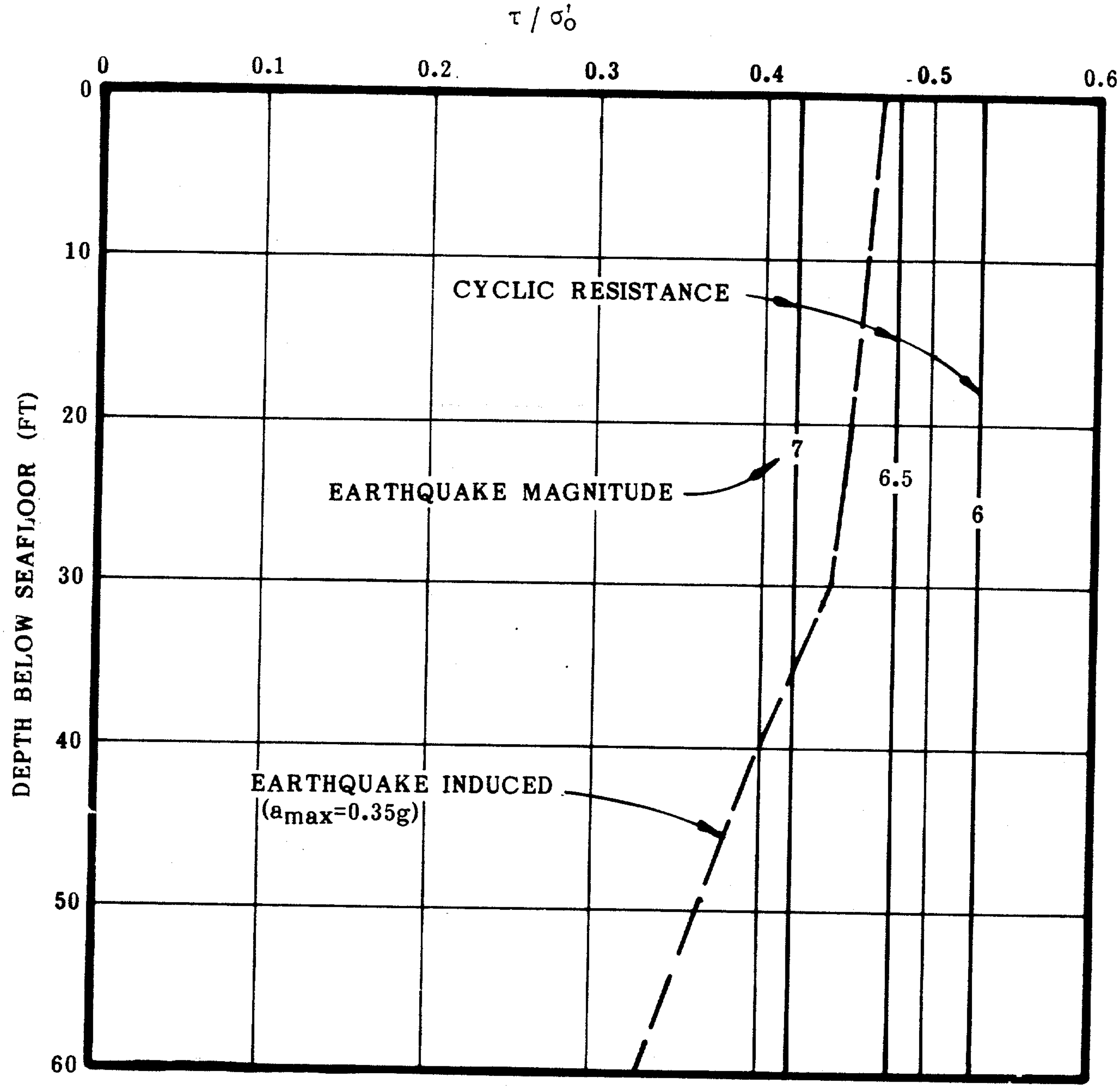


FIGURE 8.2.4 RESULTS FOR FREE-FIELD CONDITIONS, UPPER BOUND

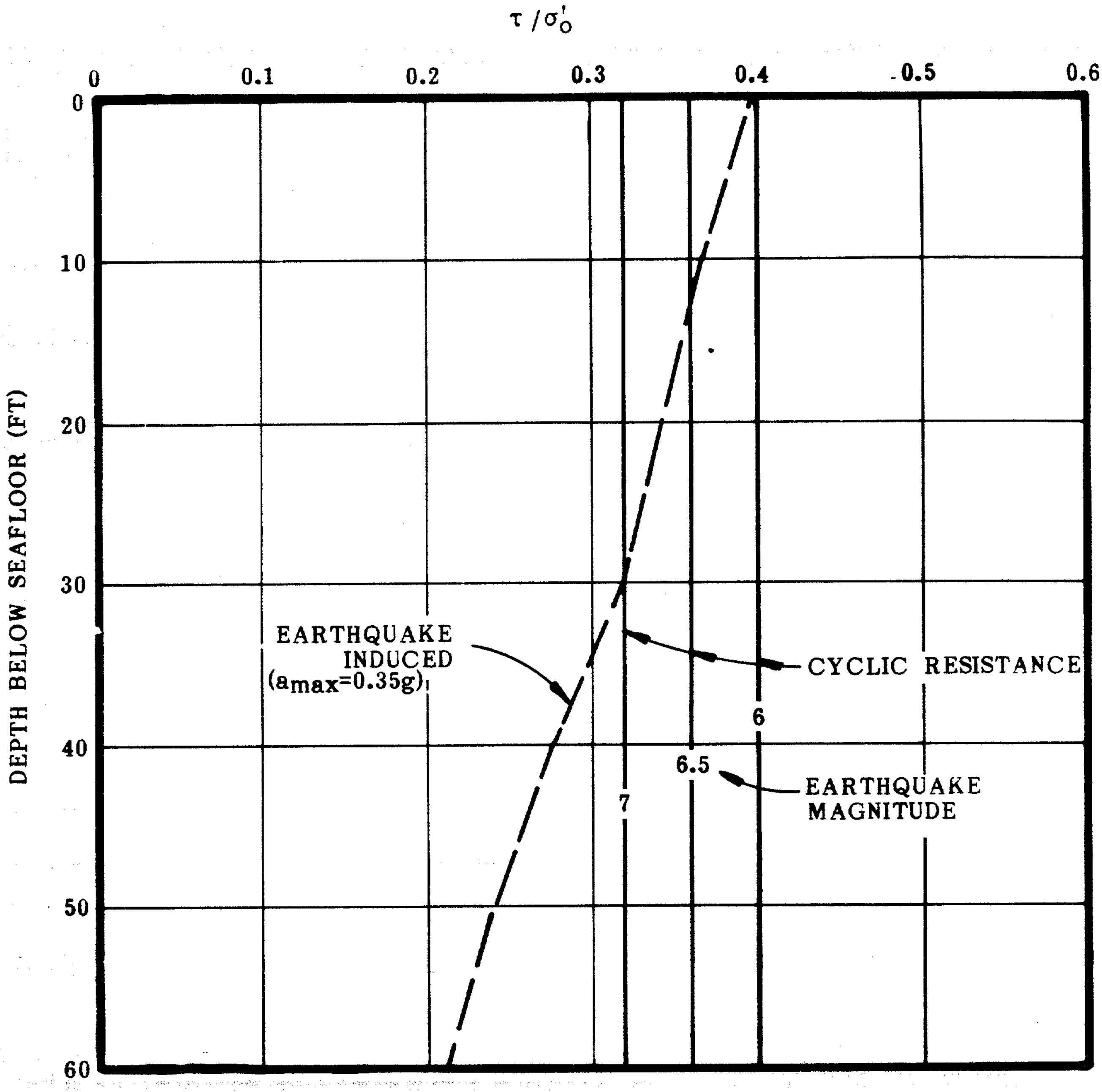


FIGURE 8.2.5 RESULTS UNDER GRAVITY STRUCTURE, LOWER BOUND

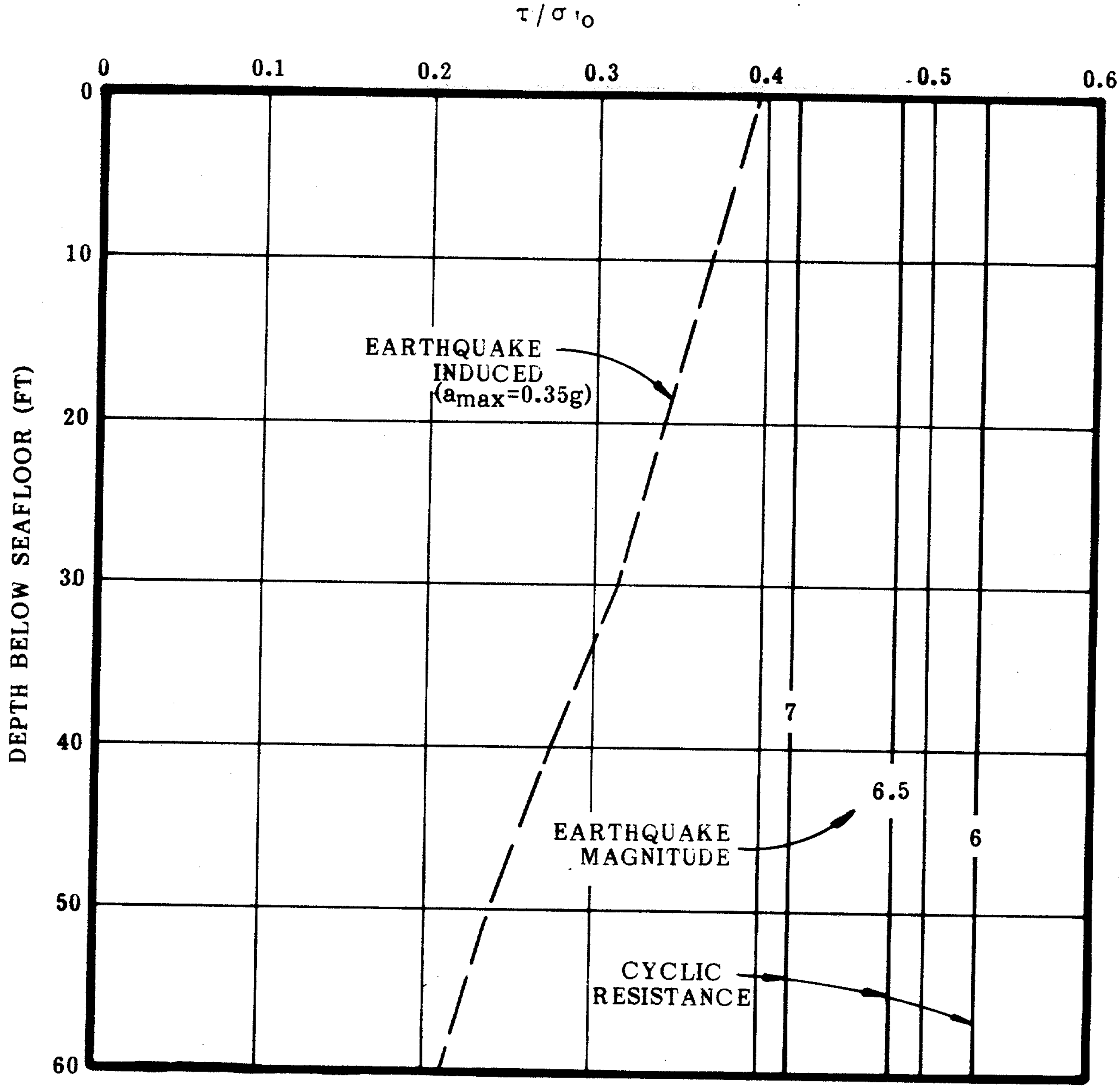


FIGURE 8.2.6 RESULTS UNDER GRAVITY STRUCTURE, UPPER BOUND

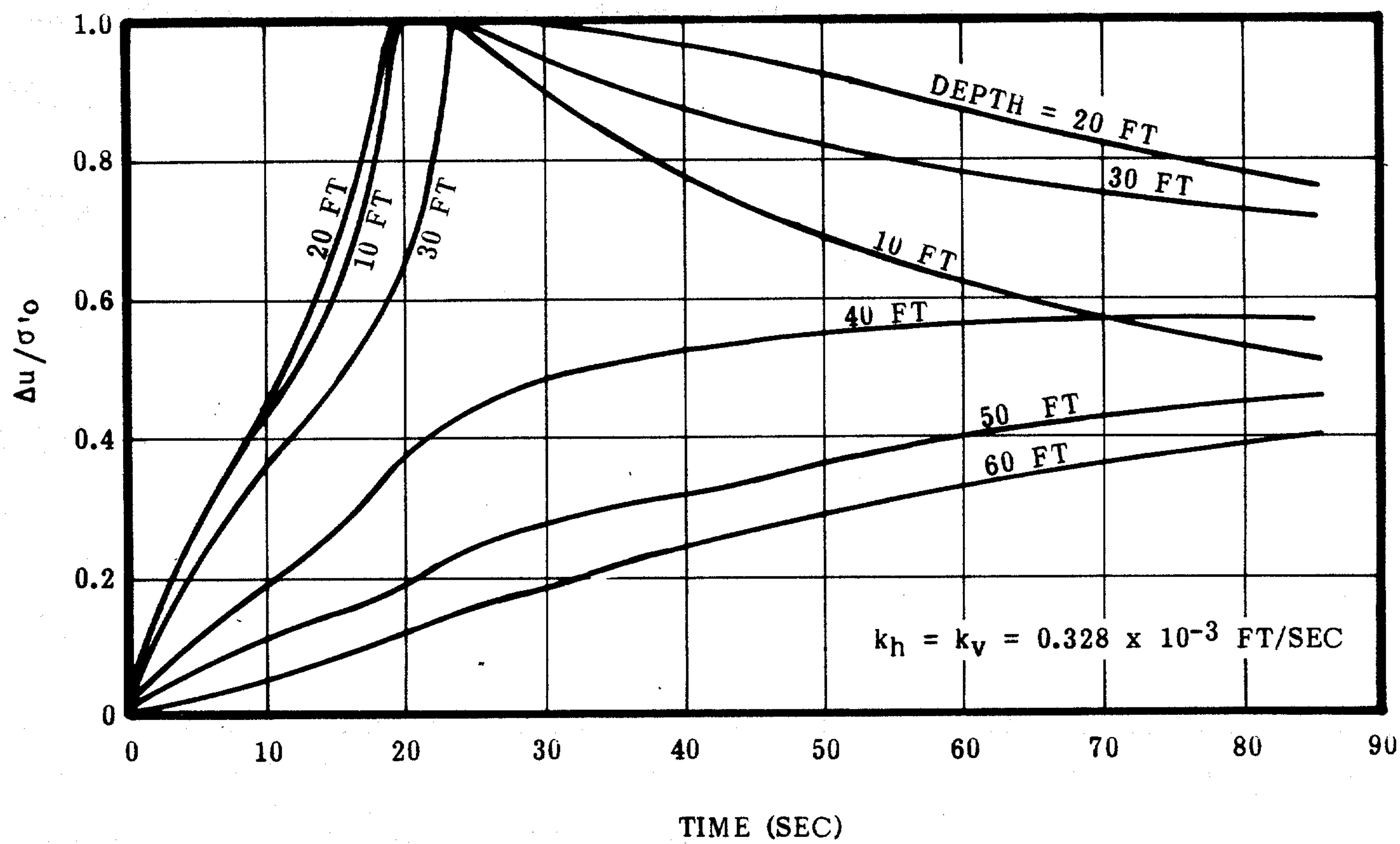


FIGURE 8.2.7 DEVELOPMENT OF EXCESS PORE PRESSURE RATIO WITH TIME, LOWER BOUND

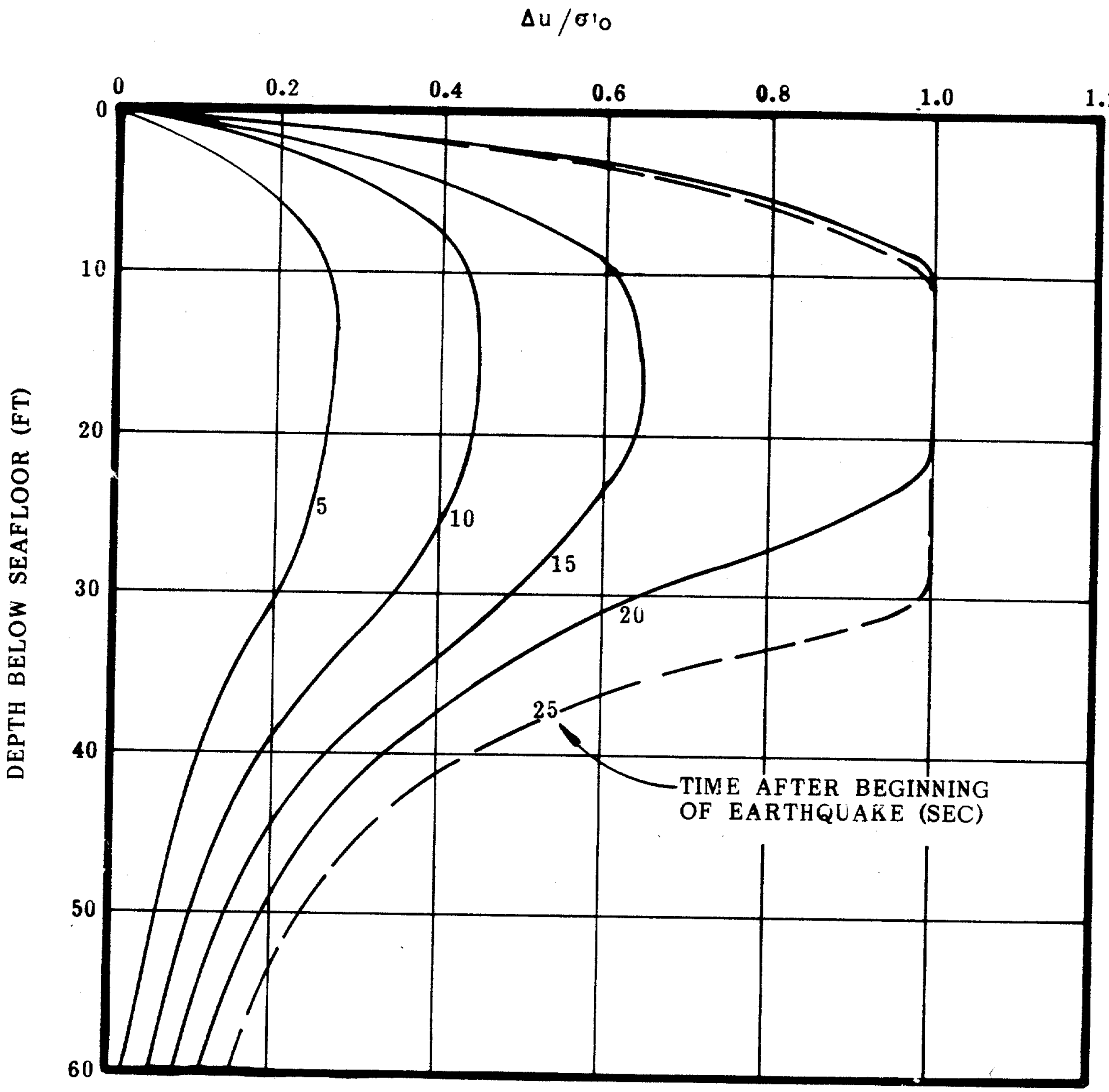


FIGURE 8.2.8 EXCESS PORE PRESSURE RATIO PROFILES, LOWER BOUND

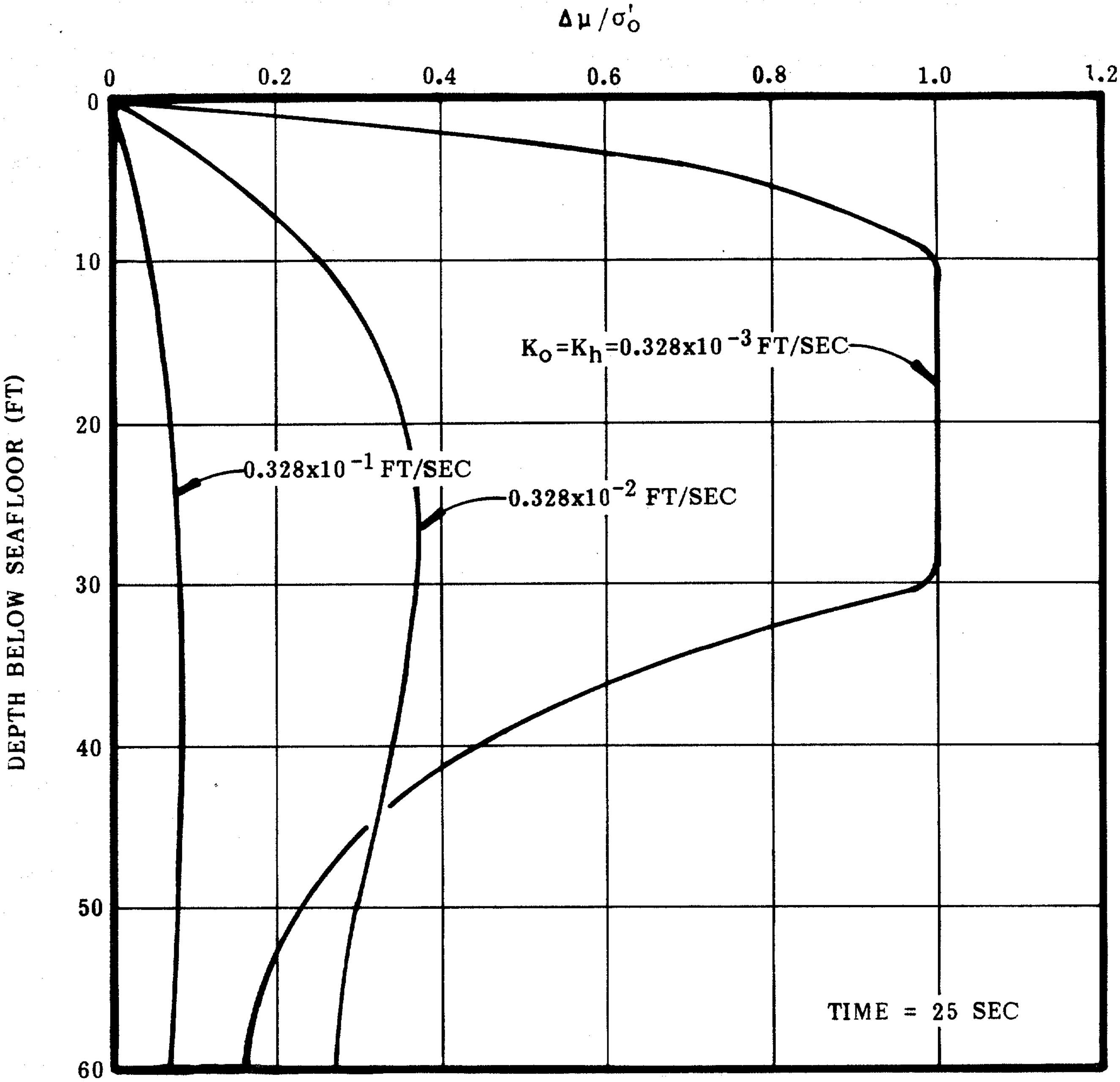
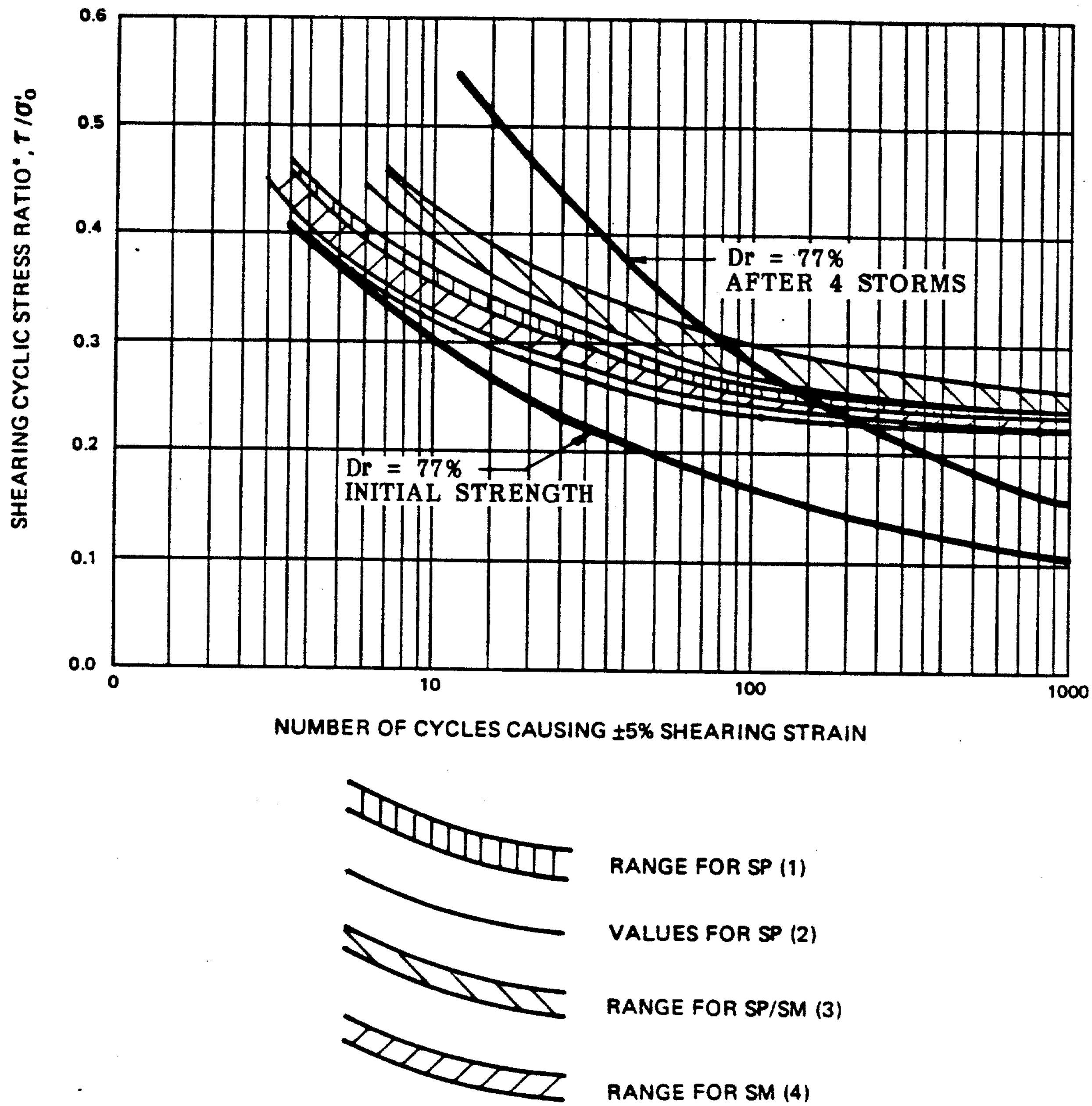


FIGURE 8.2.9 <u>EFFECT OF PERMEABILITY ON EXCESS PORE PRESSURE RATIO, LOWER BOUND</u>



*DEFINED AS RATIO OF CYCLIC SHEARING STRESS (τ) TO INITIAL EFFECTIVE VERTICAL STRESS (σ'_o)

FIGURE 8.2.10 EFFECT OF 4 STORMS AND SUBSEQUENT DISSIPATION OF EXCESS PORE PRESSURE, WITH SANDS IN NORTH ALEUTIAN SHELF

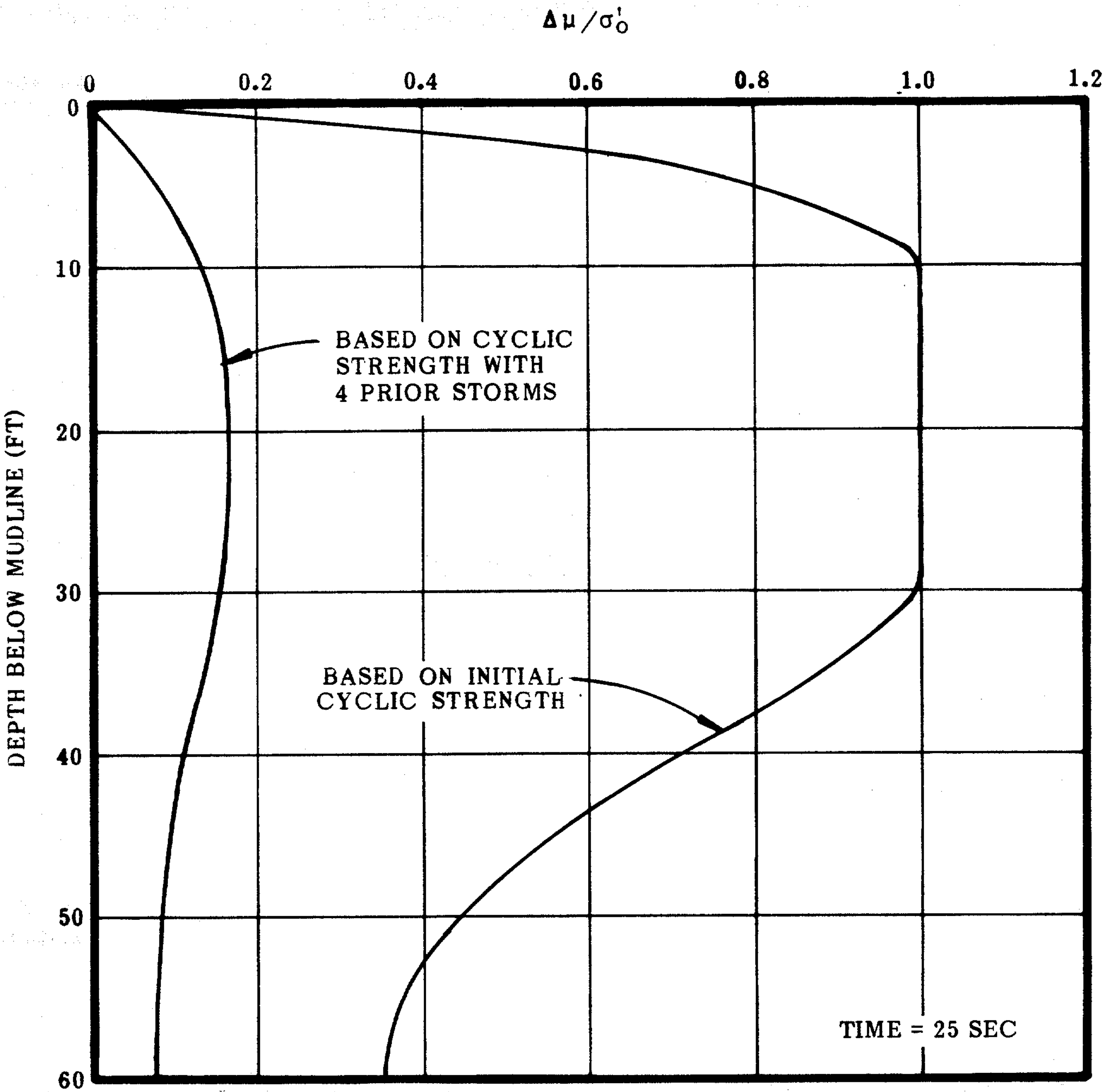


FIGURE 8.2.11 EFFECT OF PRIOR CYCLIC LOADING ON EXCESS PORE PRESSURE RATIOS (D_r = 77%)

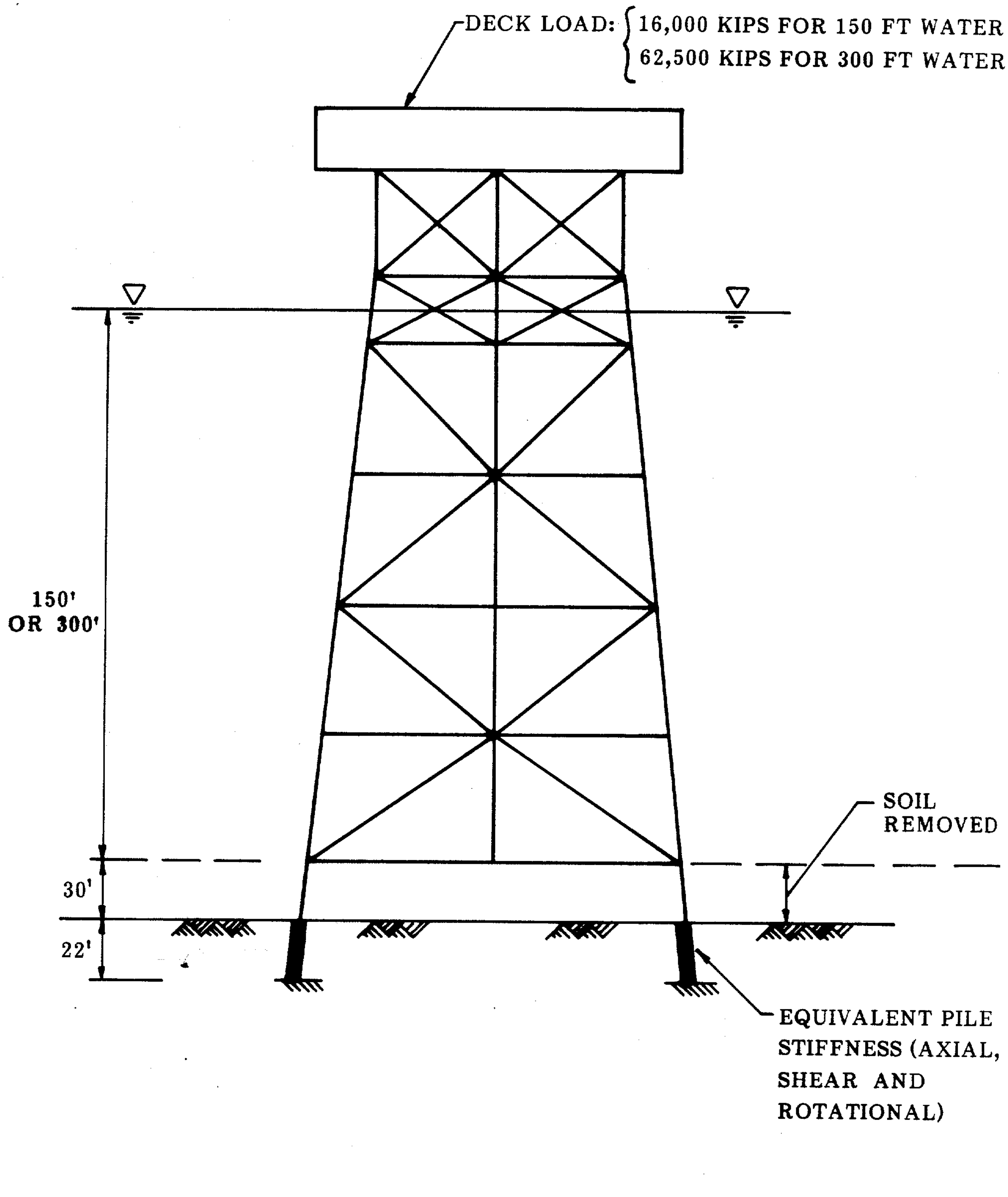
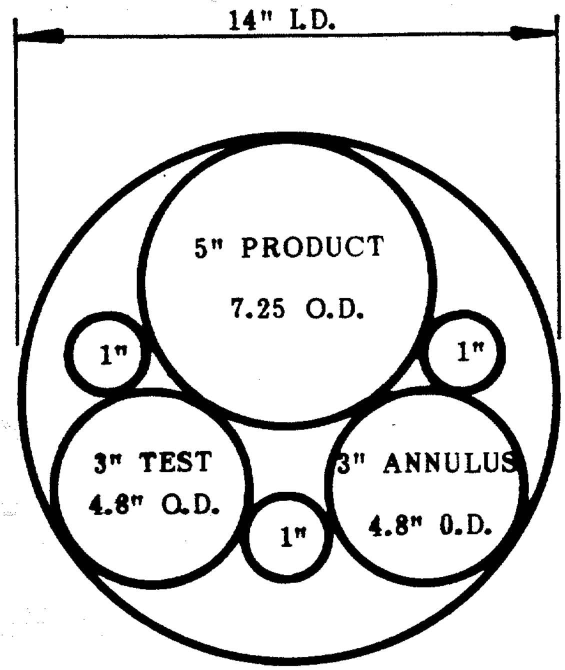
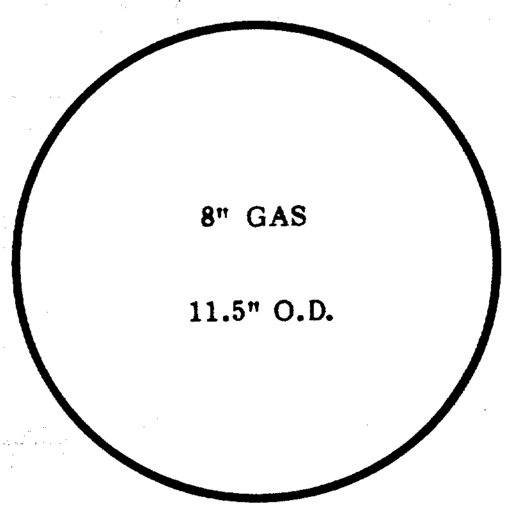


FIGURE 8.3.1 LIQUEFACTION ANALYSIS MODEL FOR THE JACKET



7 FLEXIBLE PRODUCTION RISERS



1 FLEXIBLE GAS REINJECTION RISER

FIGURE 9.2.1 <u>FLEXIBLE PRODUCTION GAS REINJECTION RISER LAYOUTS</u>

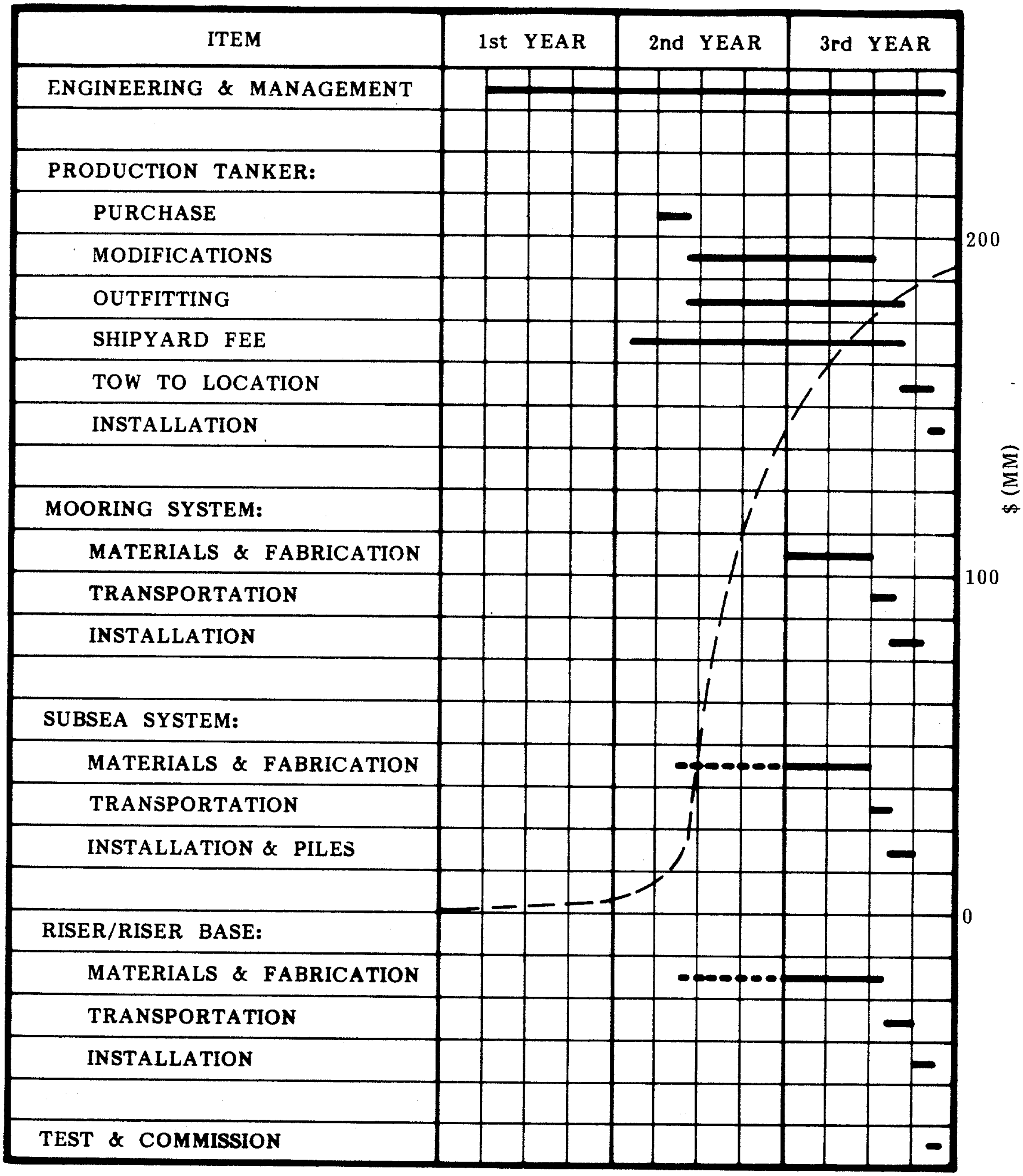


FIGURE 9.3.1 SCHEDULE AND CUMULATIVE CASH FLOW FOR FPSO SYSTEM FABRICATED IN JAPAN, 100,000 BOPD

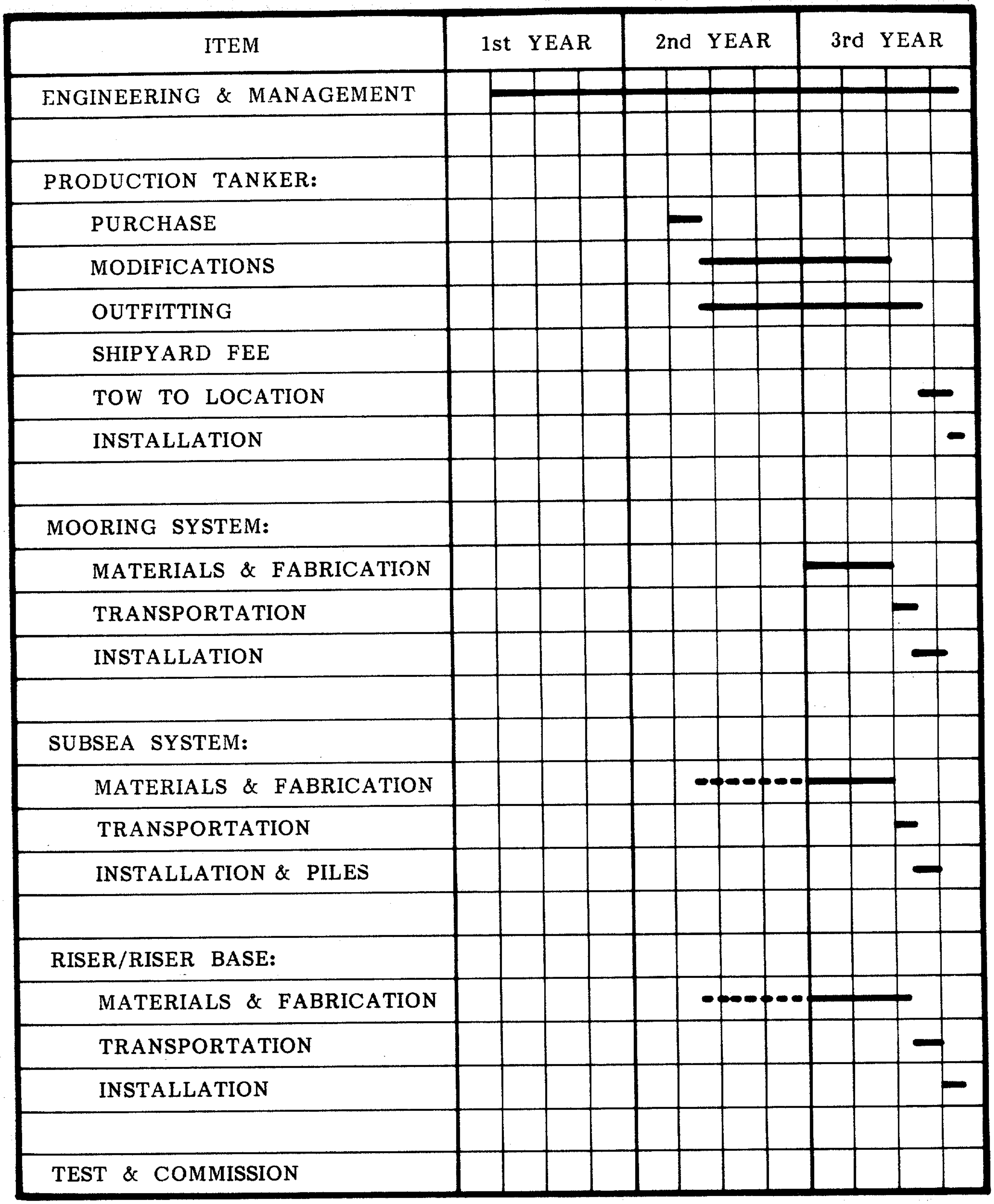


FIGURE 9.3.2 <u>SCHEDULE FOR FSO SYSTEM FABRICATED IN JAPAN, 100,000 BOPD</u>

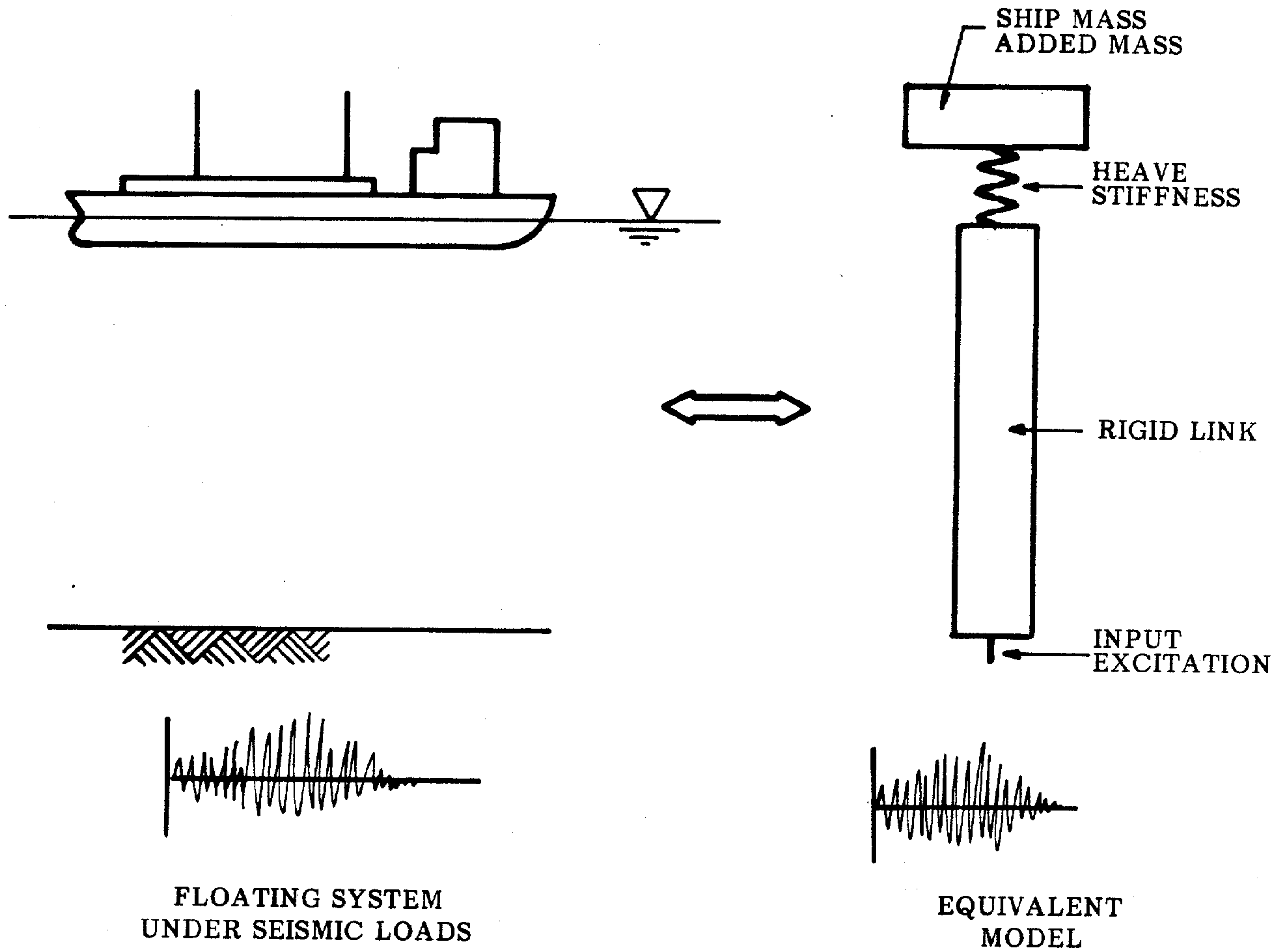


FIGURE 9.5.1 SIMPLIFIED ANALYSIS MODEL FOR EFFECTS OF SEAQUAKES ON FLOATING VESSELS

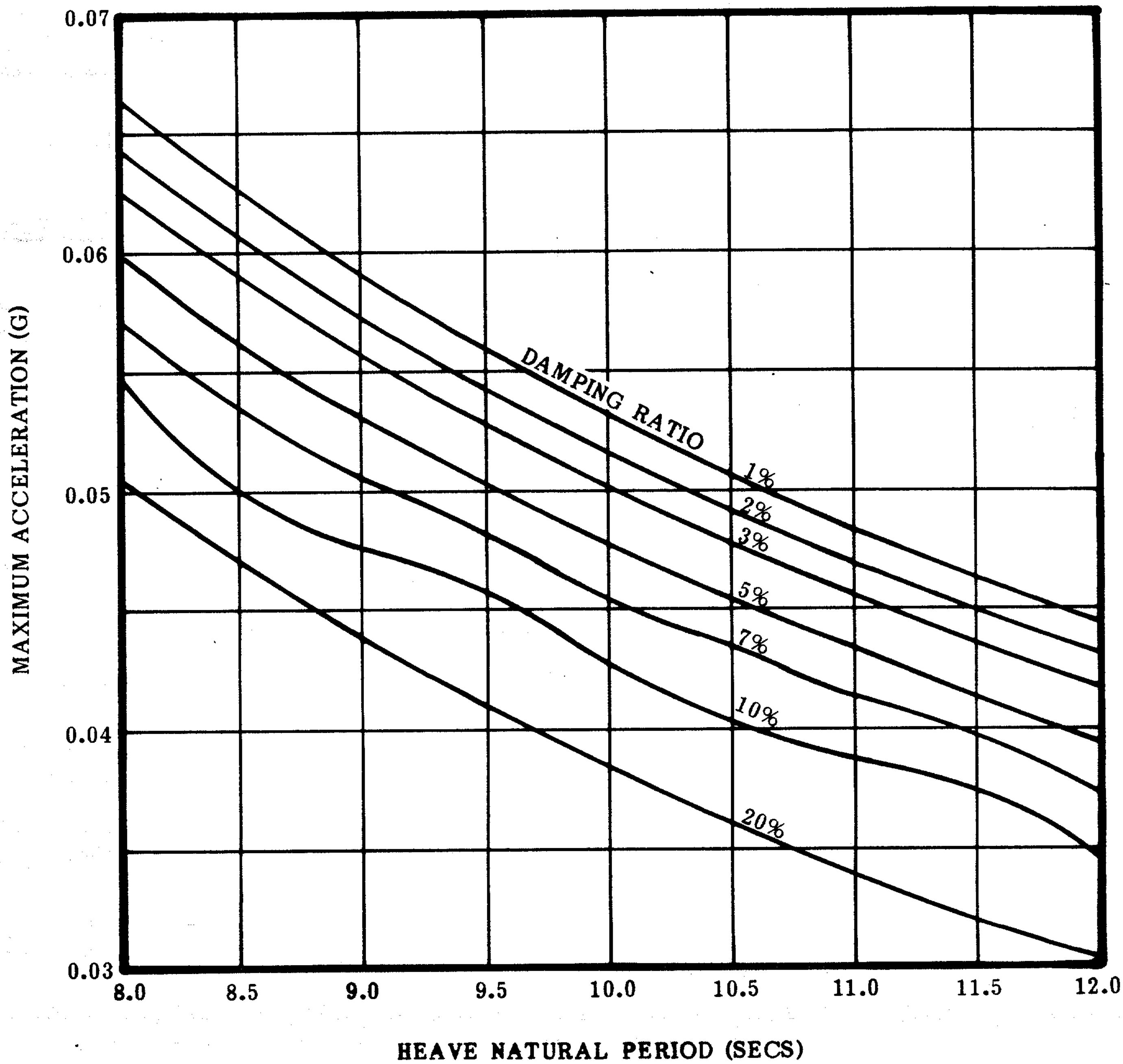


FIGURE 9.5.2 MAXIMUM ACCELERATIONS UNDER THE DESIGN EARTHQUAKE ASSUMED TO BE APPLIED AS A VERTICAL ACCELERATION

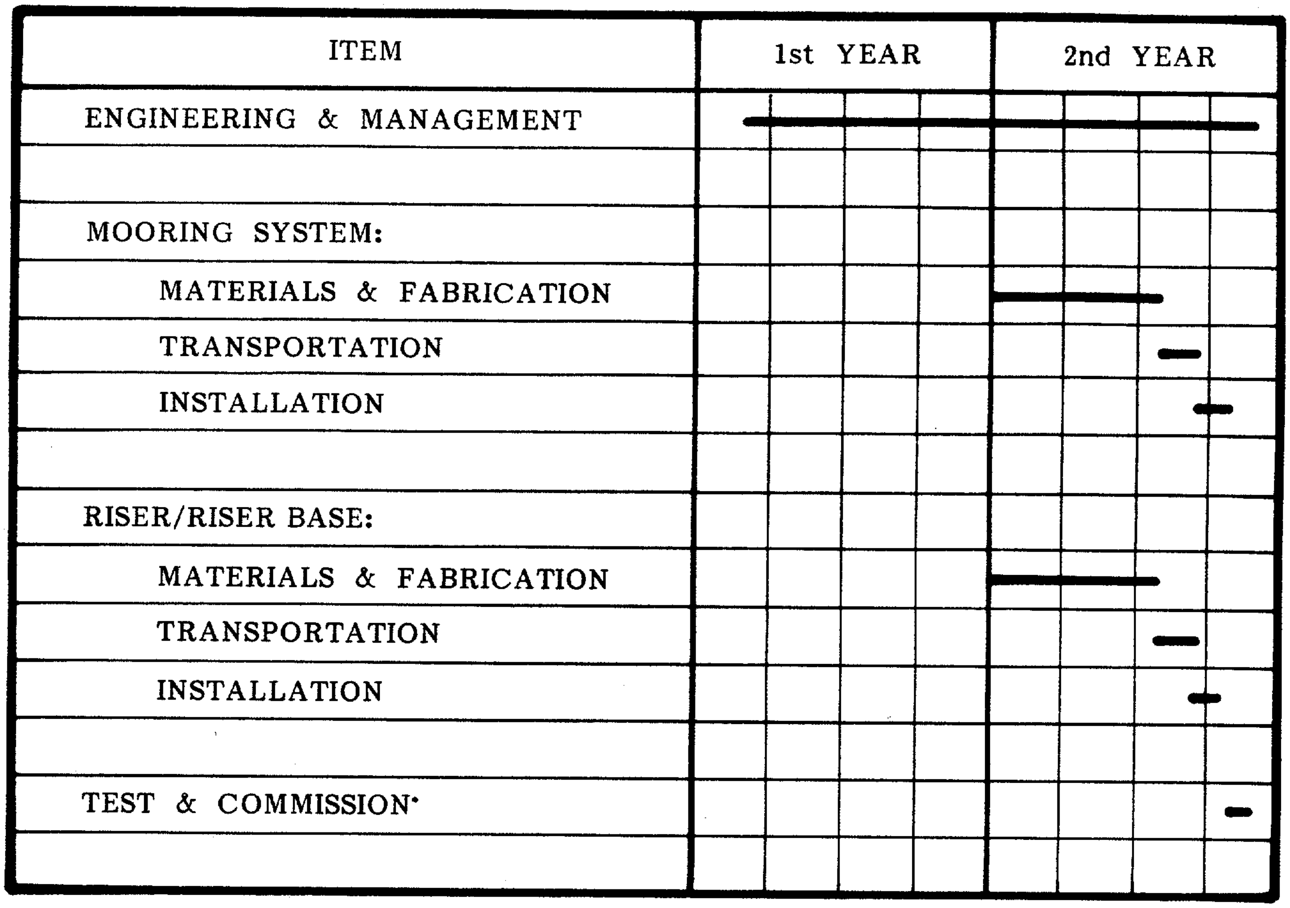


FIGURE 10.3.1 <u>REMOTE LOADING BUOY SCHEDULE</u>

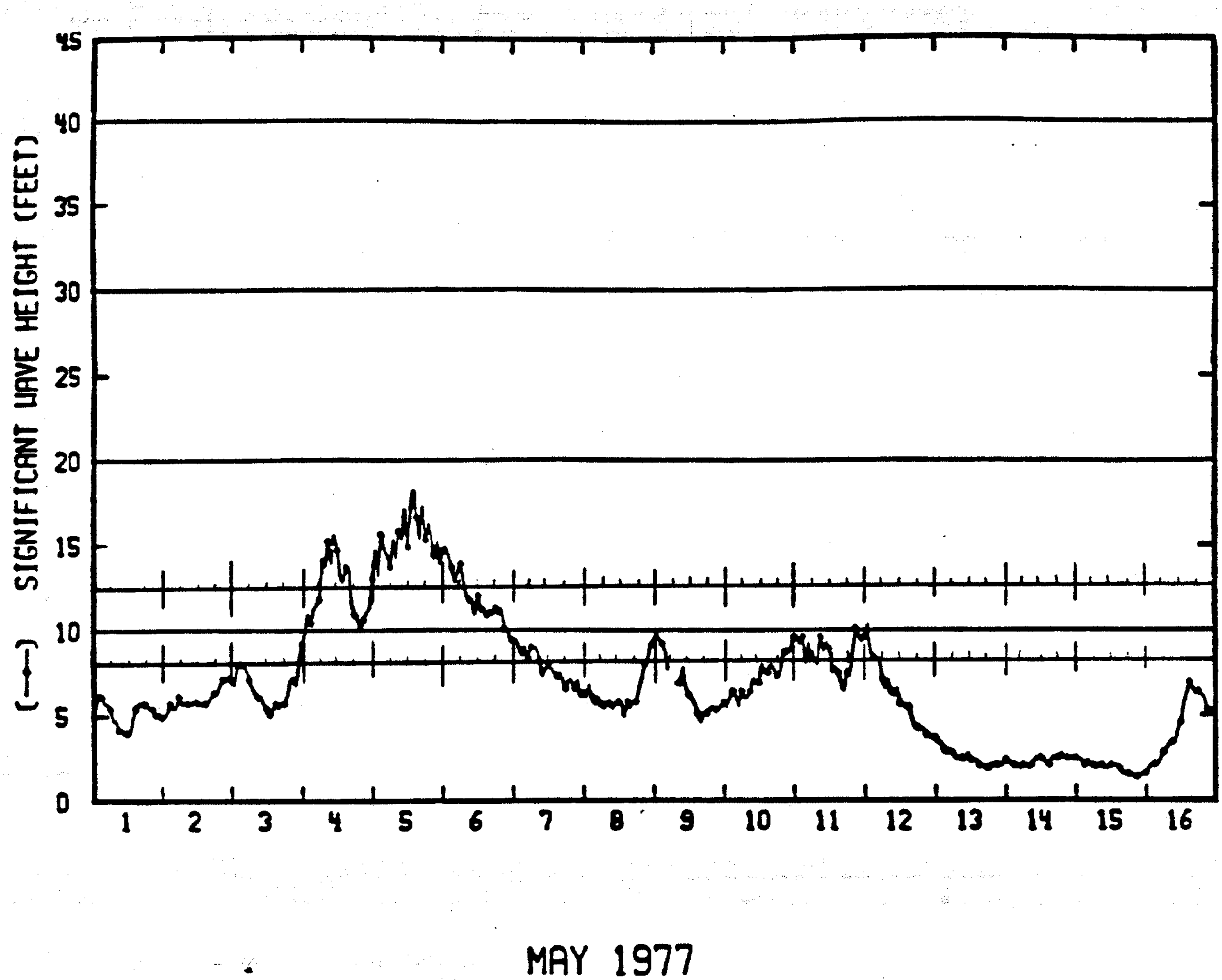


FIGURE 10.4.1 TYPICAL WAVE TRACE

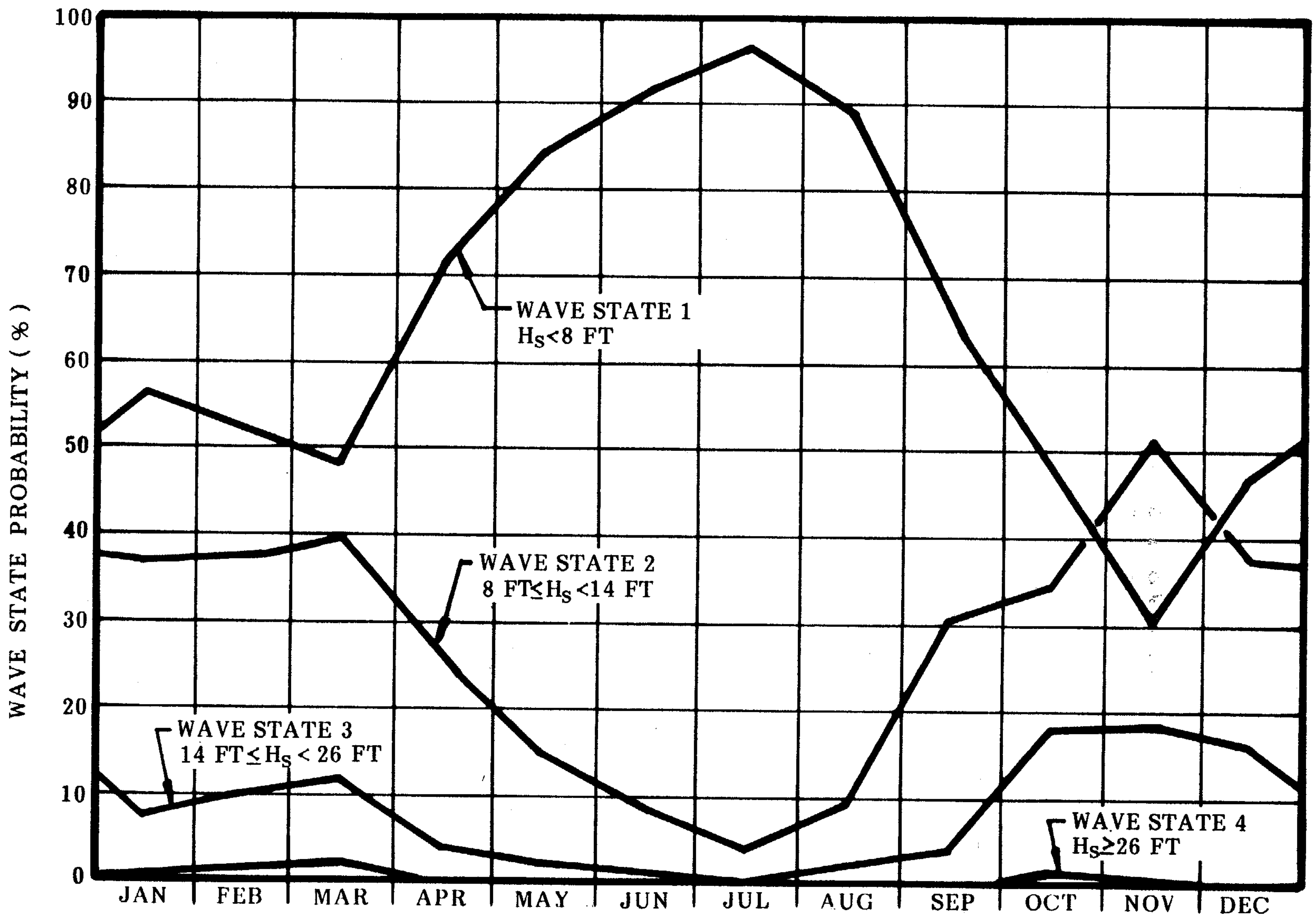


FIGURE 10.4.2 <u>MONTHLY WAVE STATE PROBABILITIES FROM WAVE PERIODS</u>

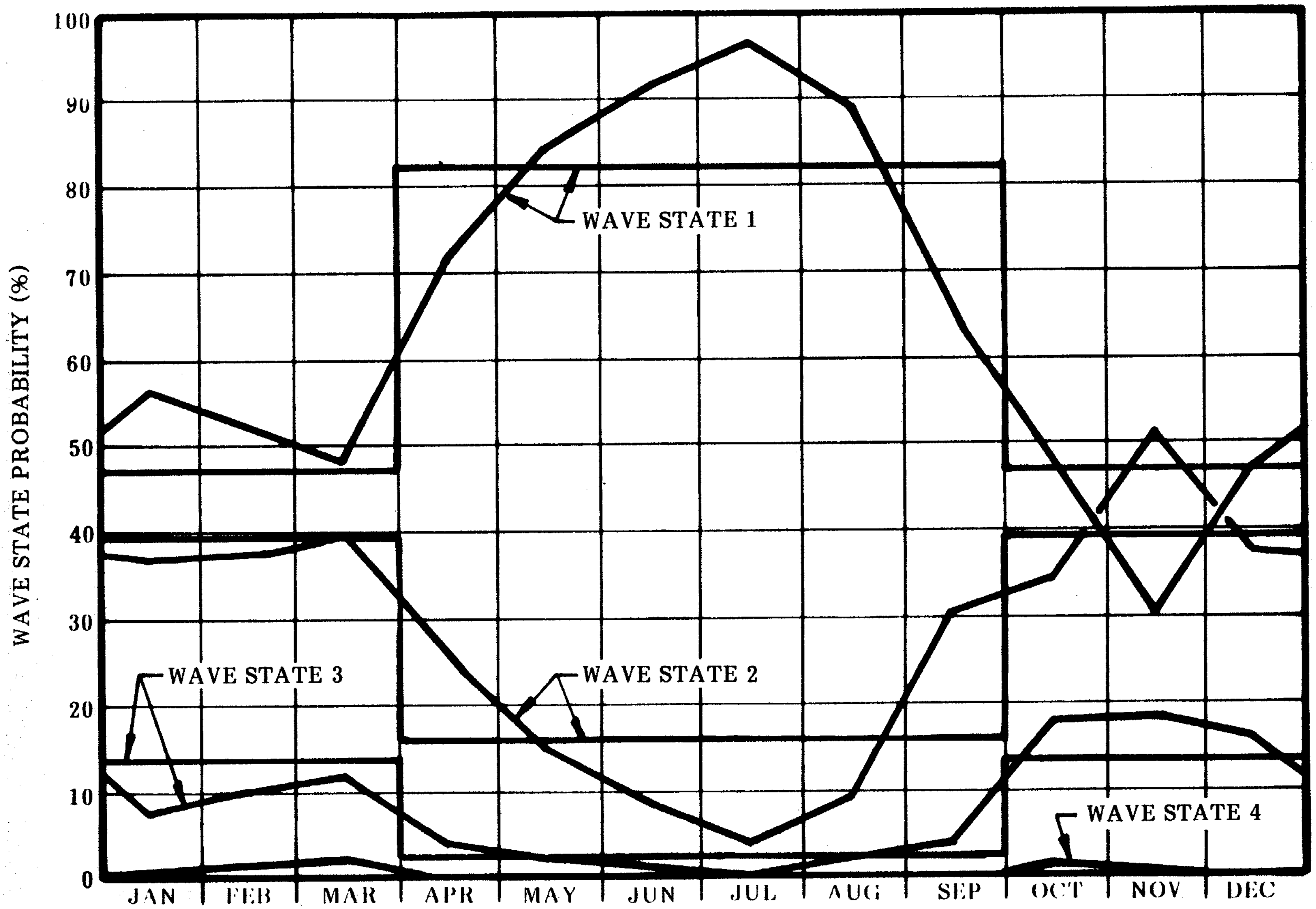


FIGURE 10.4.3 IDEALIZED WAVE STATE PROBABILITIES

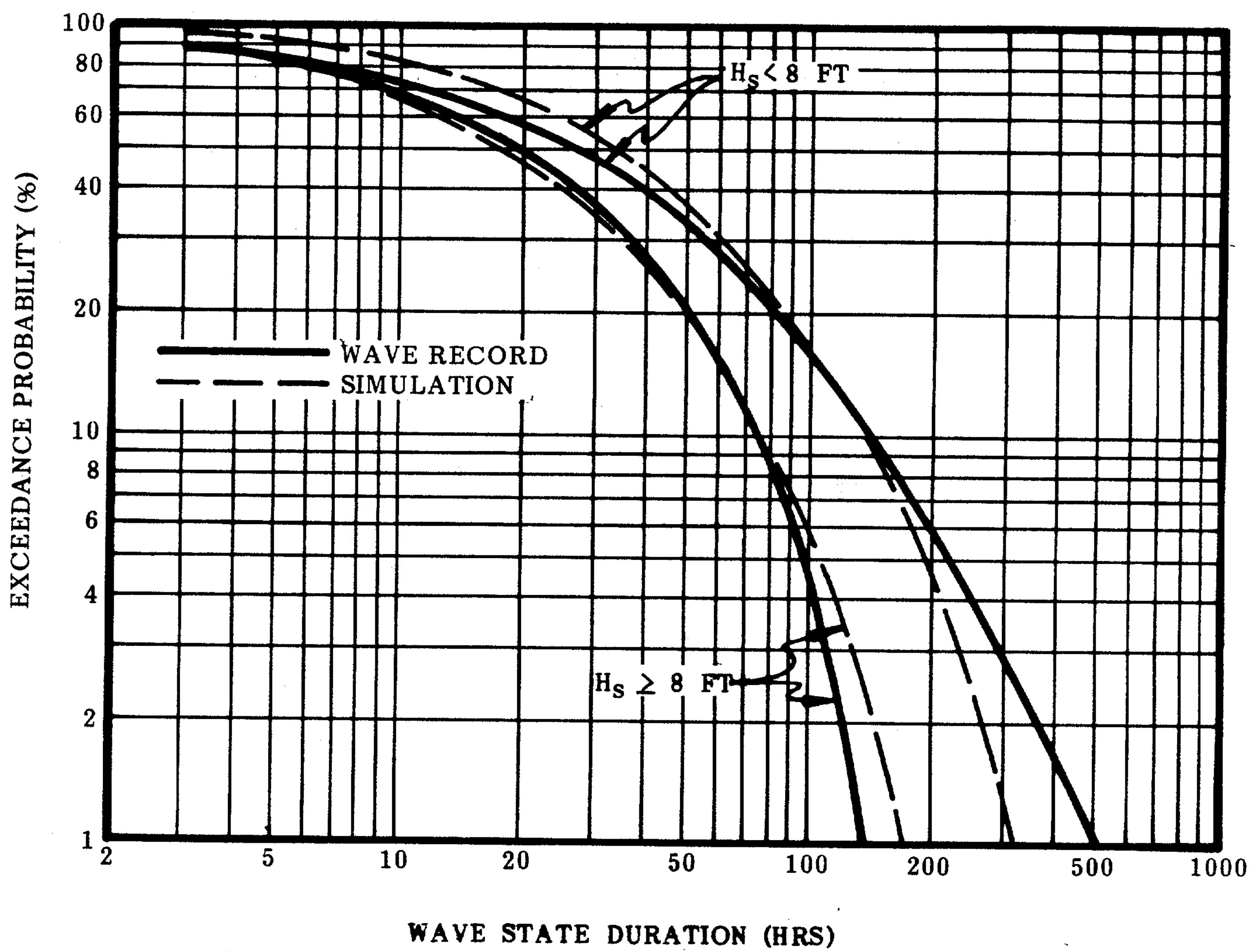


FIGURE 10.4.4 WAVE STATE DURATION DISTRIBUTIONS FOR WAVE STATE 1 AND FOR COMBINED WAVE STATES 2, 3, AND 4

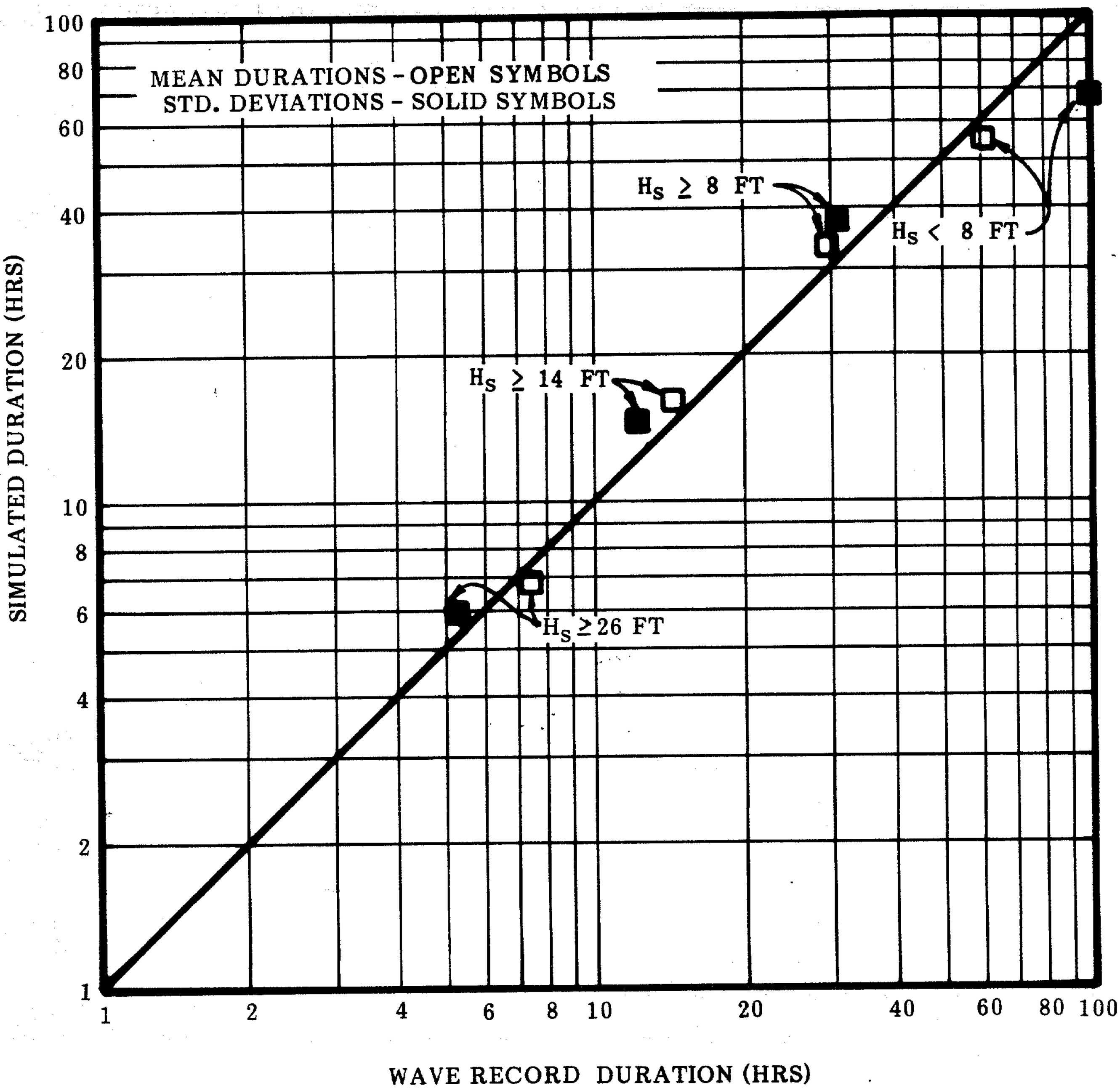


FIGURE 10.4.5 A COMPARISON BETWEEN SIMULATED MEAN DURATIONS OF WAVE STATES AND MEAN DURATIONS FROM THE WAVE RECORD

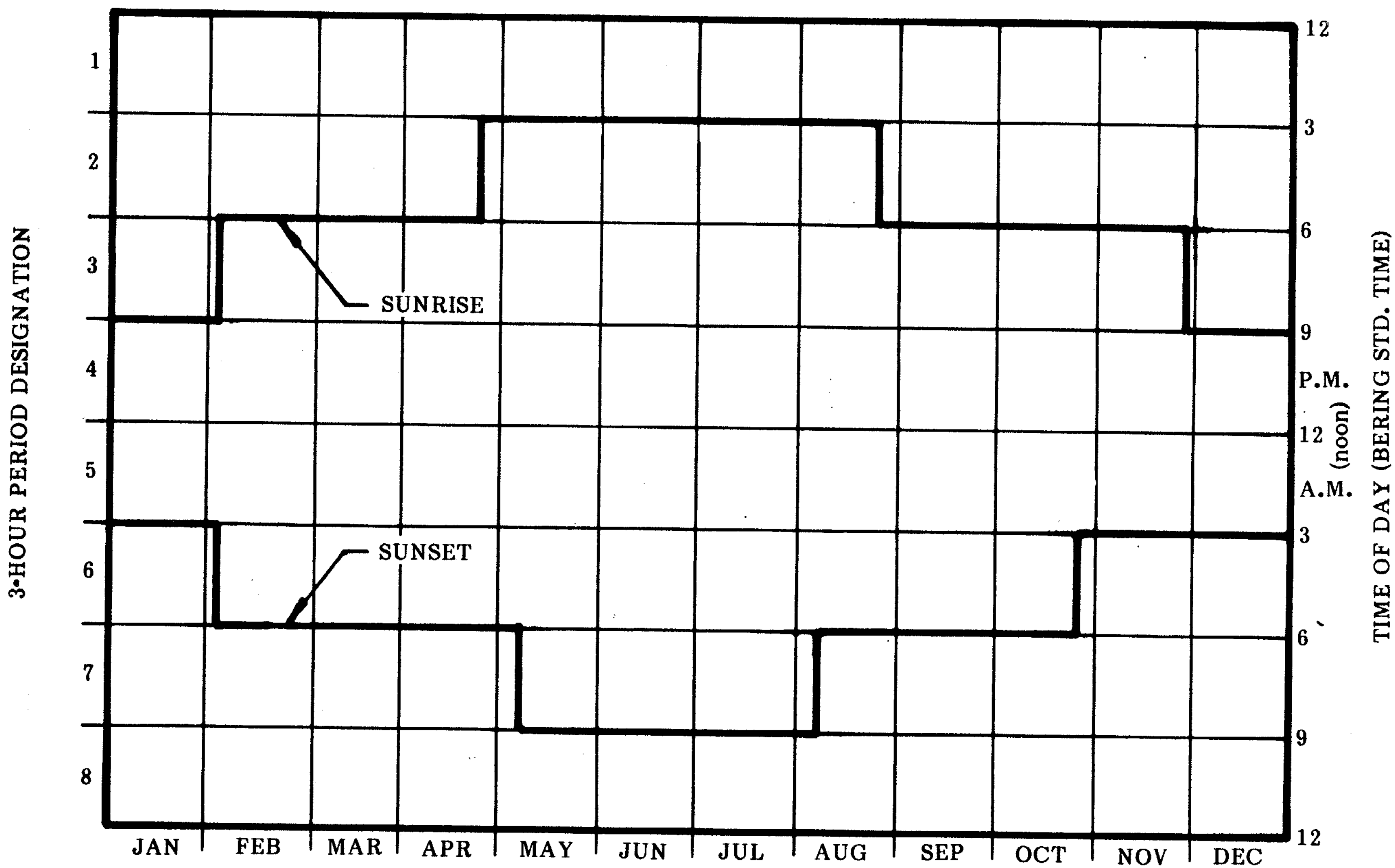


FIGURE 10.4.6 TIME PERIODS OF DAYLIGHT VERSUS TIME OF YEAR

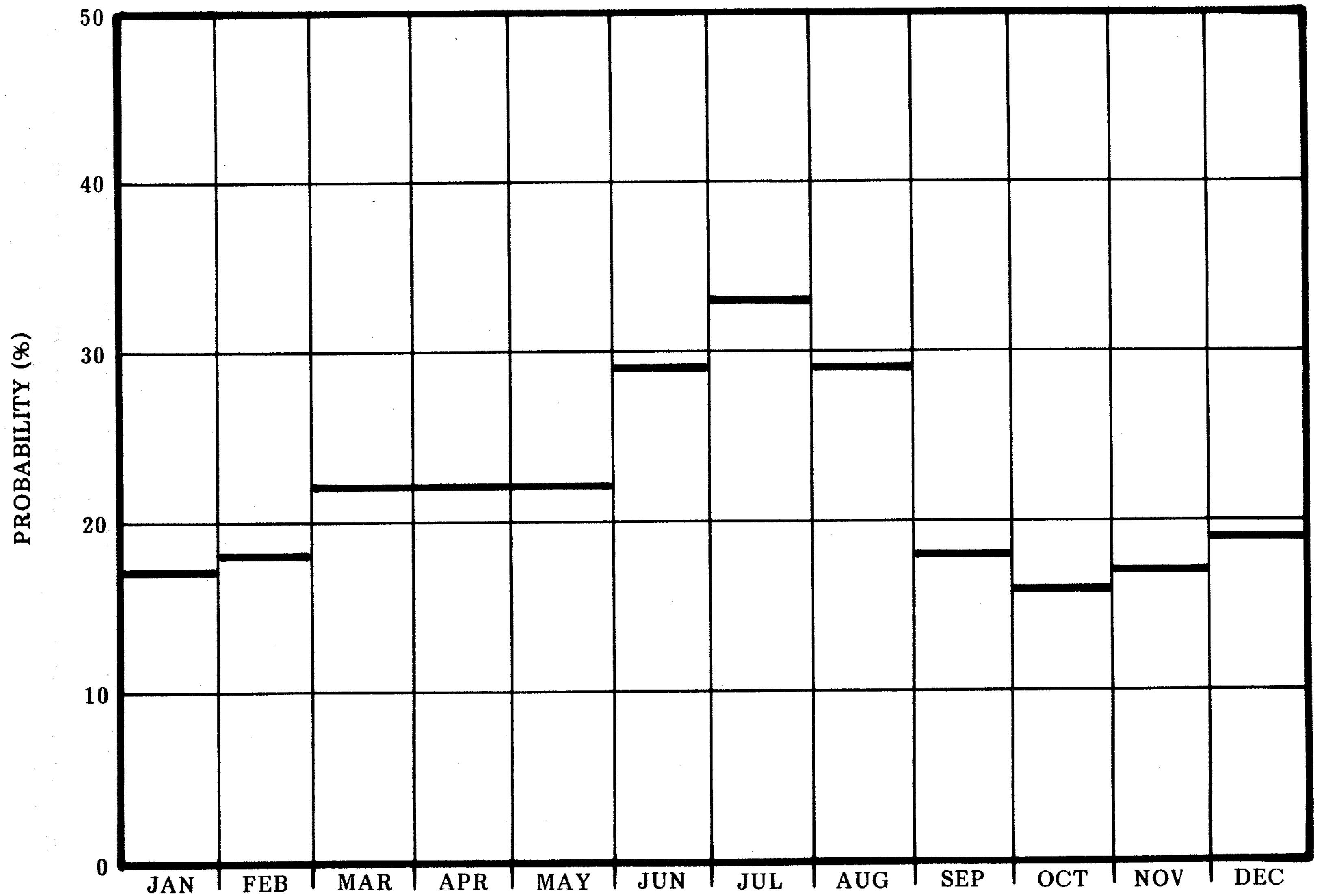


FIGURE 10.4.7 PROBABILITIES OF VISIBILITY LESS THAN 2 N. MI. AT PORT MOLLER

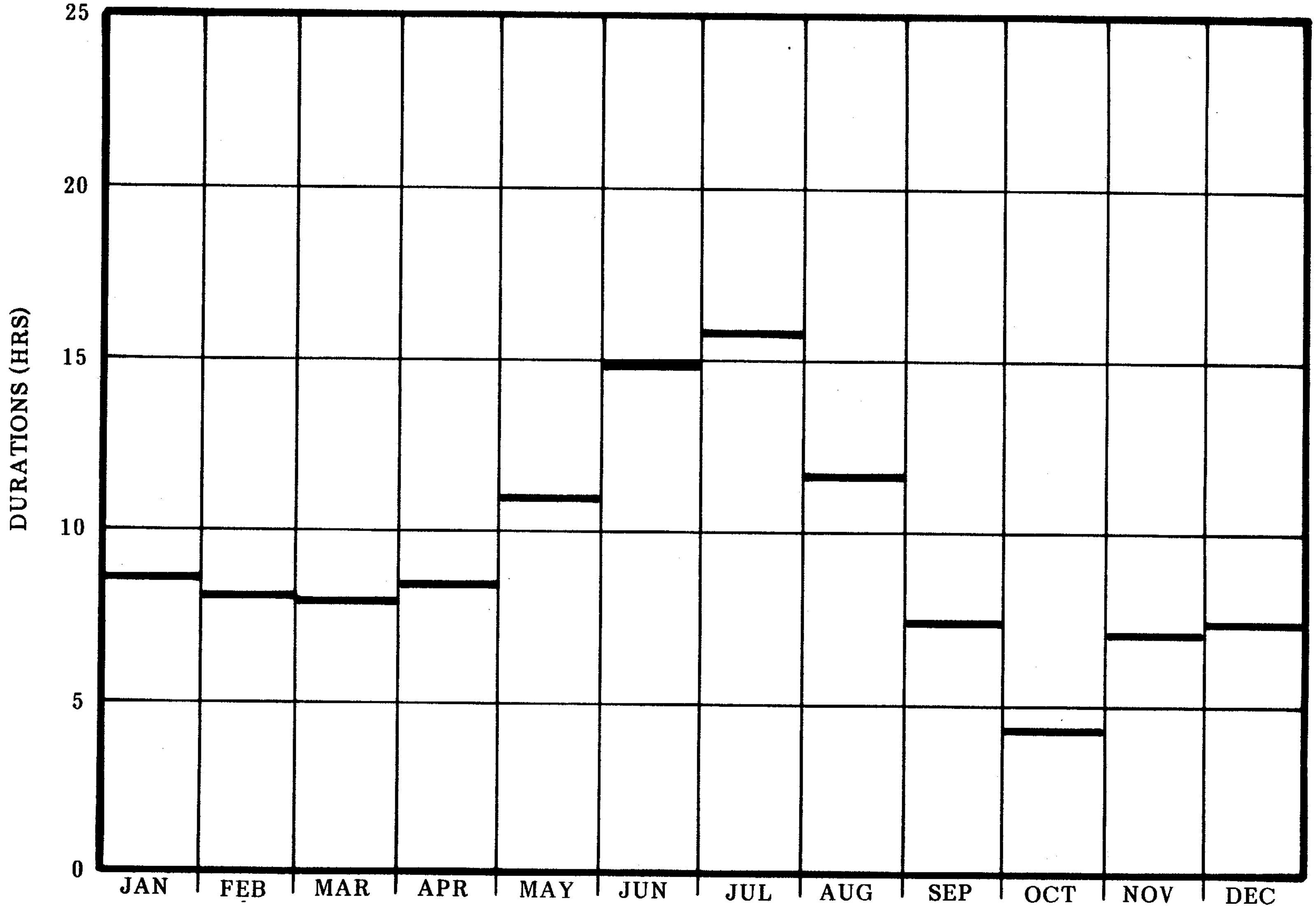


FIGURE 10.4.8 MEAN DURATION OF VISIBILITY LESS THAN 2 N. MI. AT PORT MOLLER

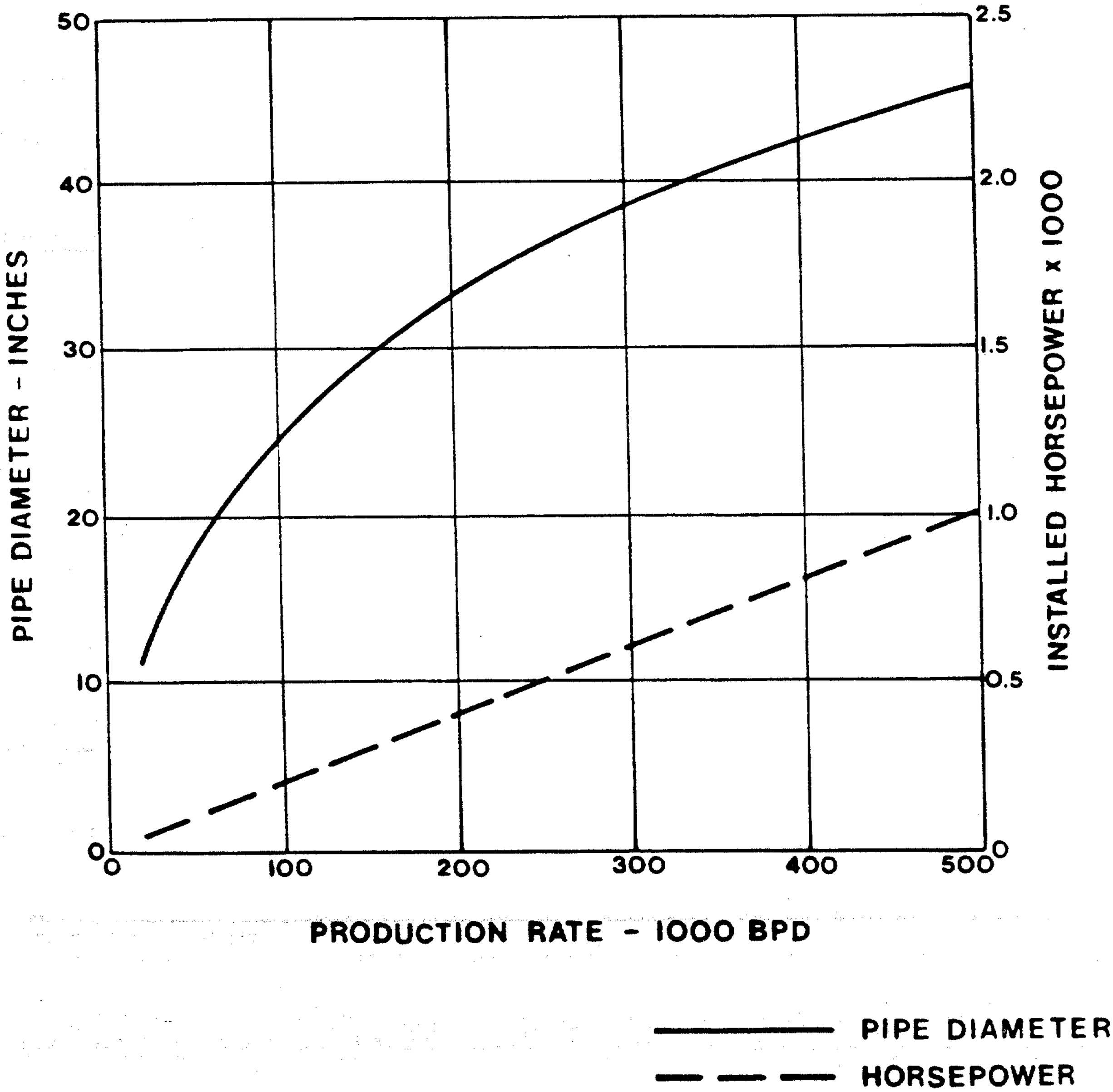


FIGURE 11.1.1 OFFSHORE PIPELINE FROM STEEL JACKET TO FPSO

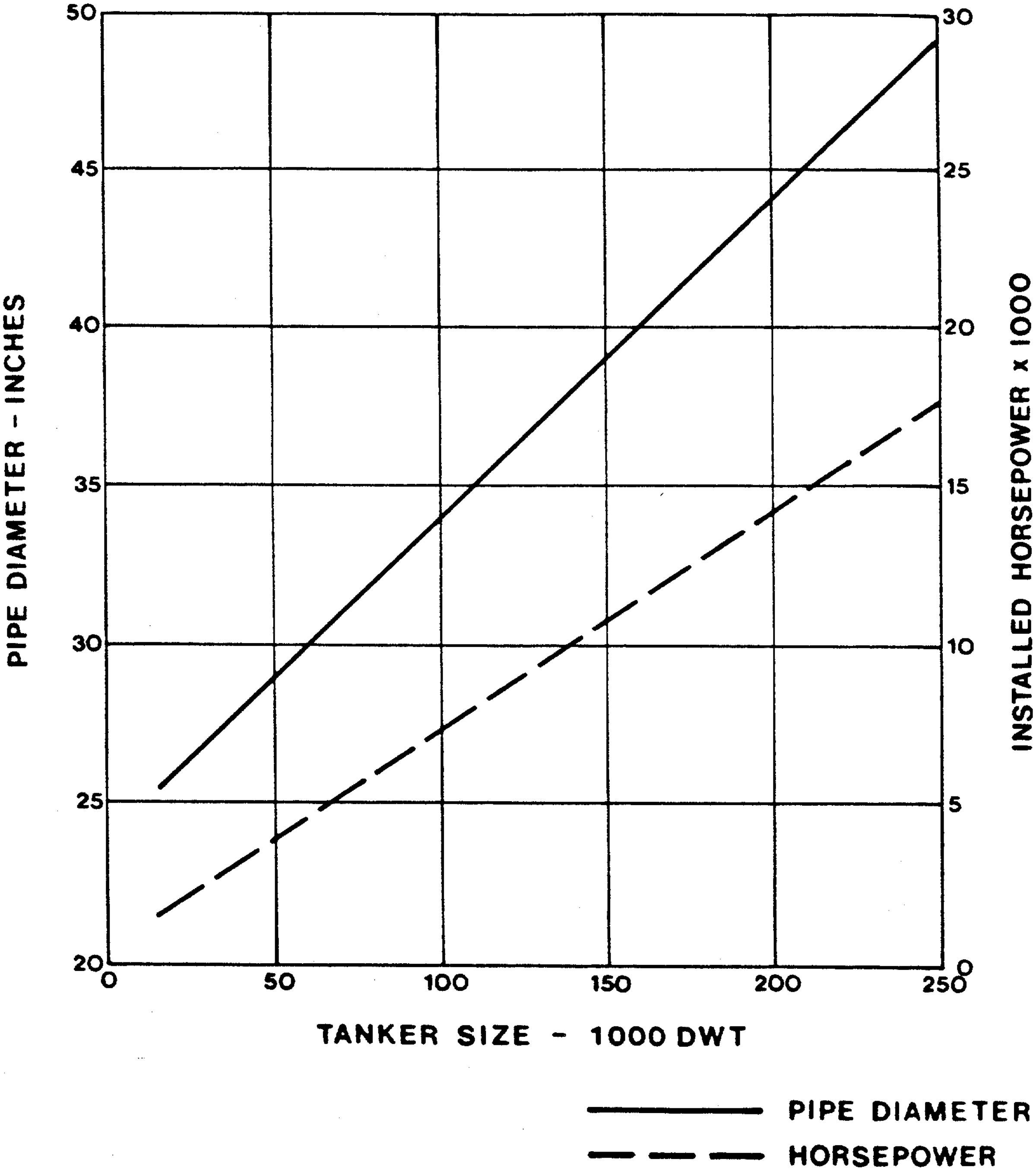


FIGURE 11.1.2 OFFSHORE PIPELINE FROM HYBRID OR CONCRETE GRAVITY PLATFORM TO FPSO

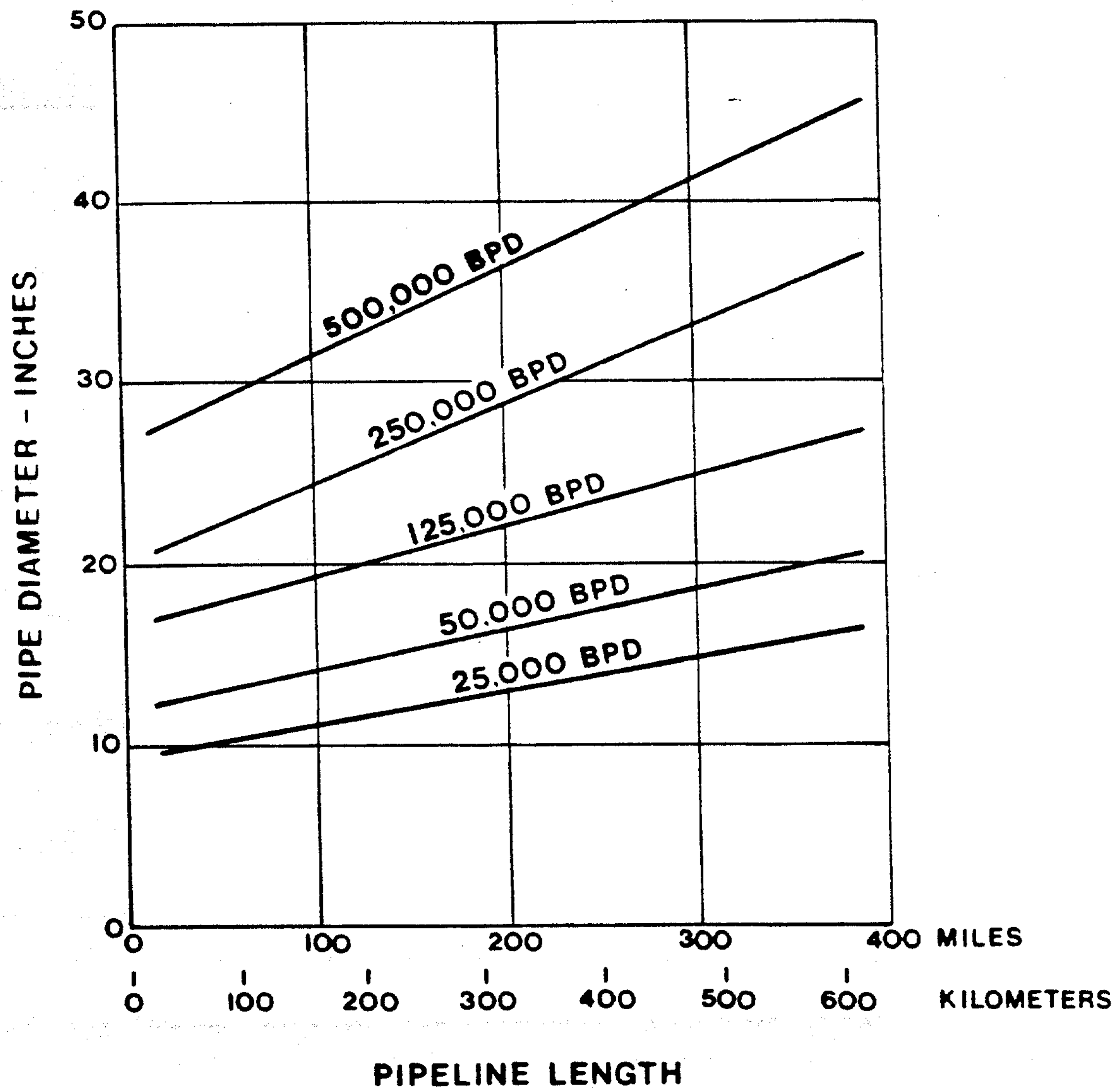


FIGURE 11.1.3 OFFSHORE PIPELINE FROM STEEL JACKET TO SHORE

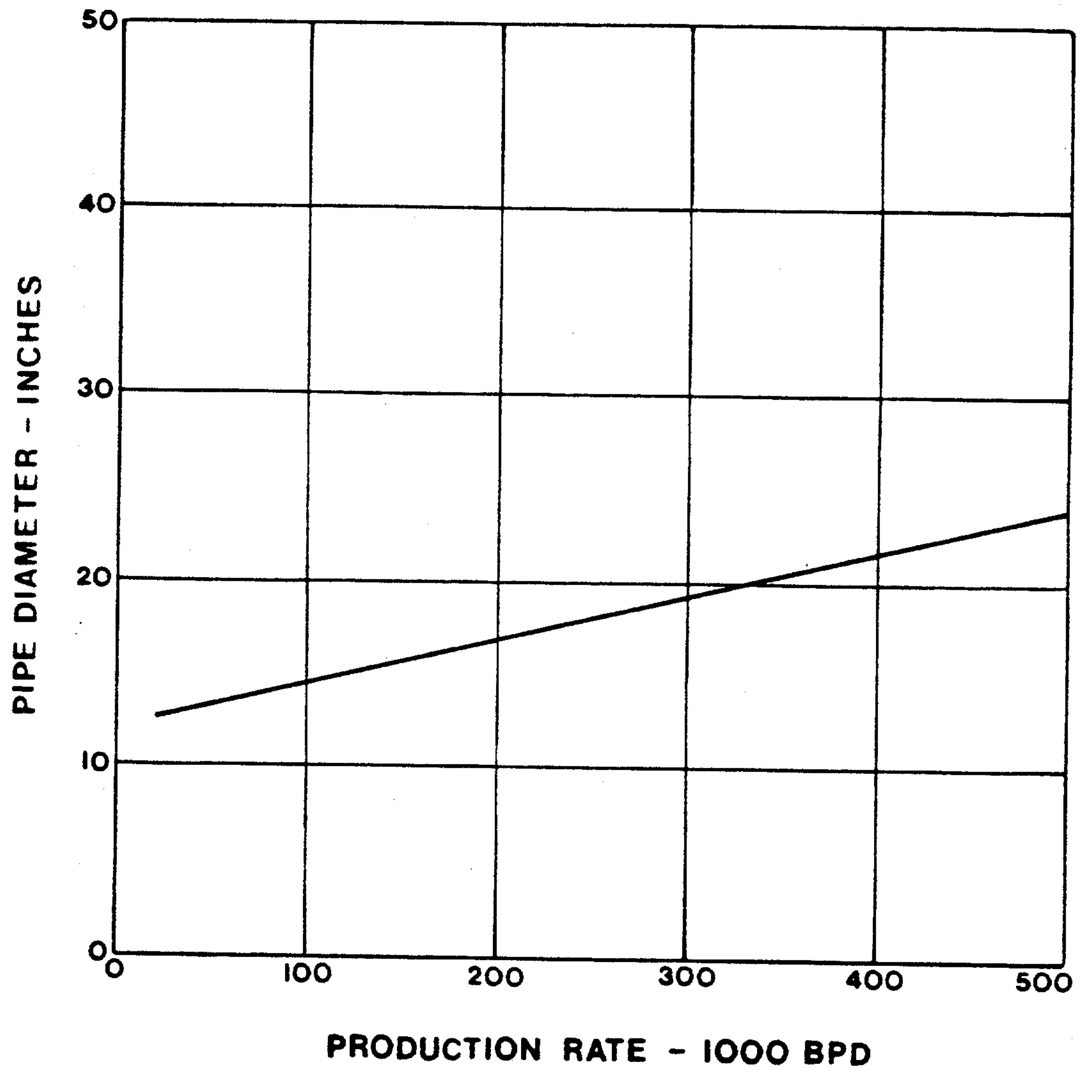


FIGURE 11.1.4 LAND PIPELINE

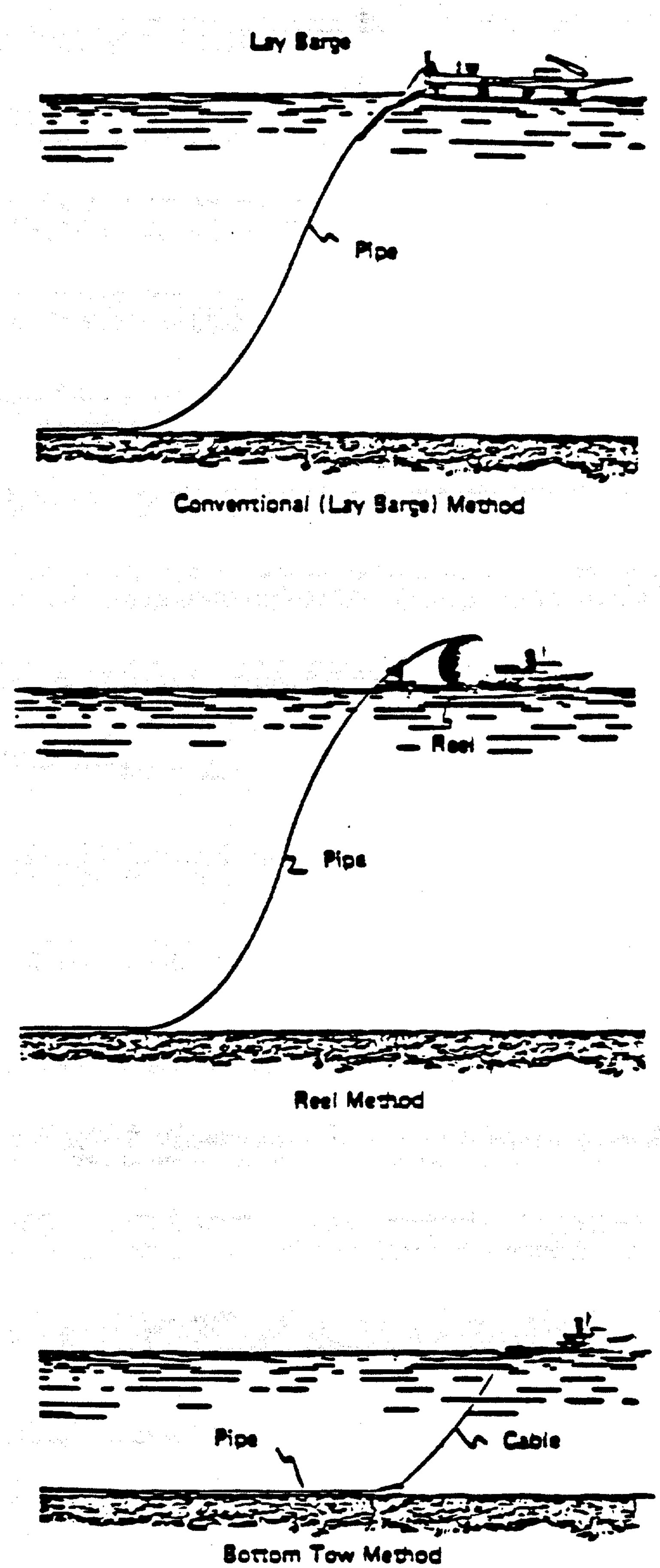
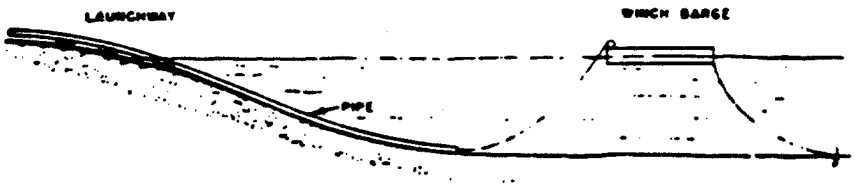
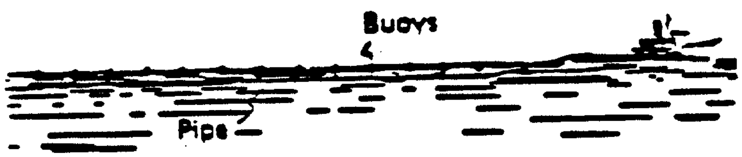


FIGURE 11.1.5 MARINE PIPELINE INSTALLATION METHODS - PART 1



Bottom Pull Method



Surface Tow

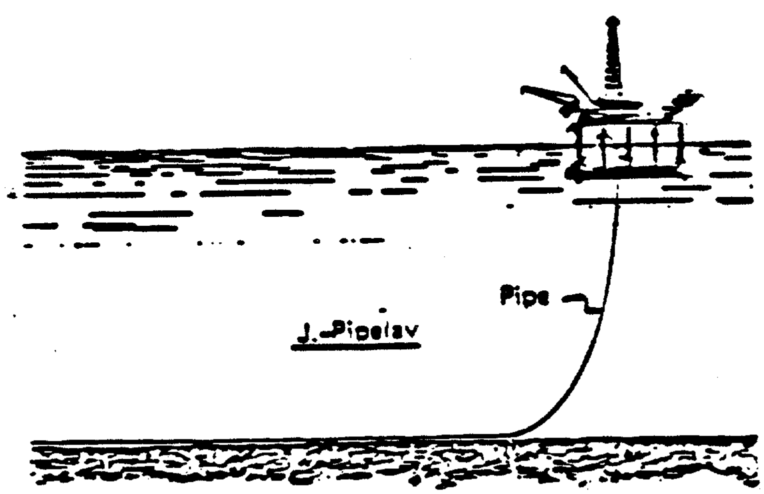


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FIGURE 11.1.6 MARINE PIPELINE INSTALLATION METHODS - PART 2

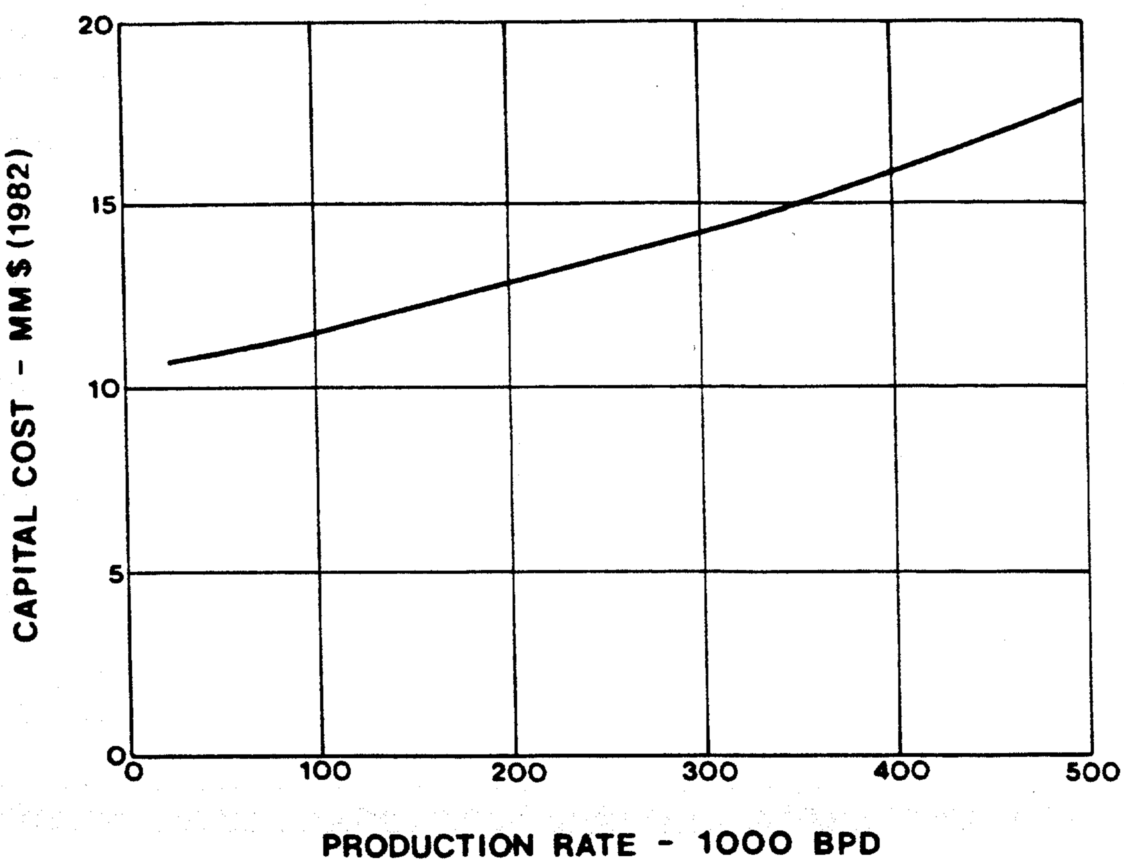


FIGURE 11.2.1 PRODUCTION PLATFORM TO FSO PIPELINE CAPITAL COST

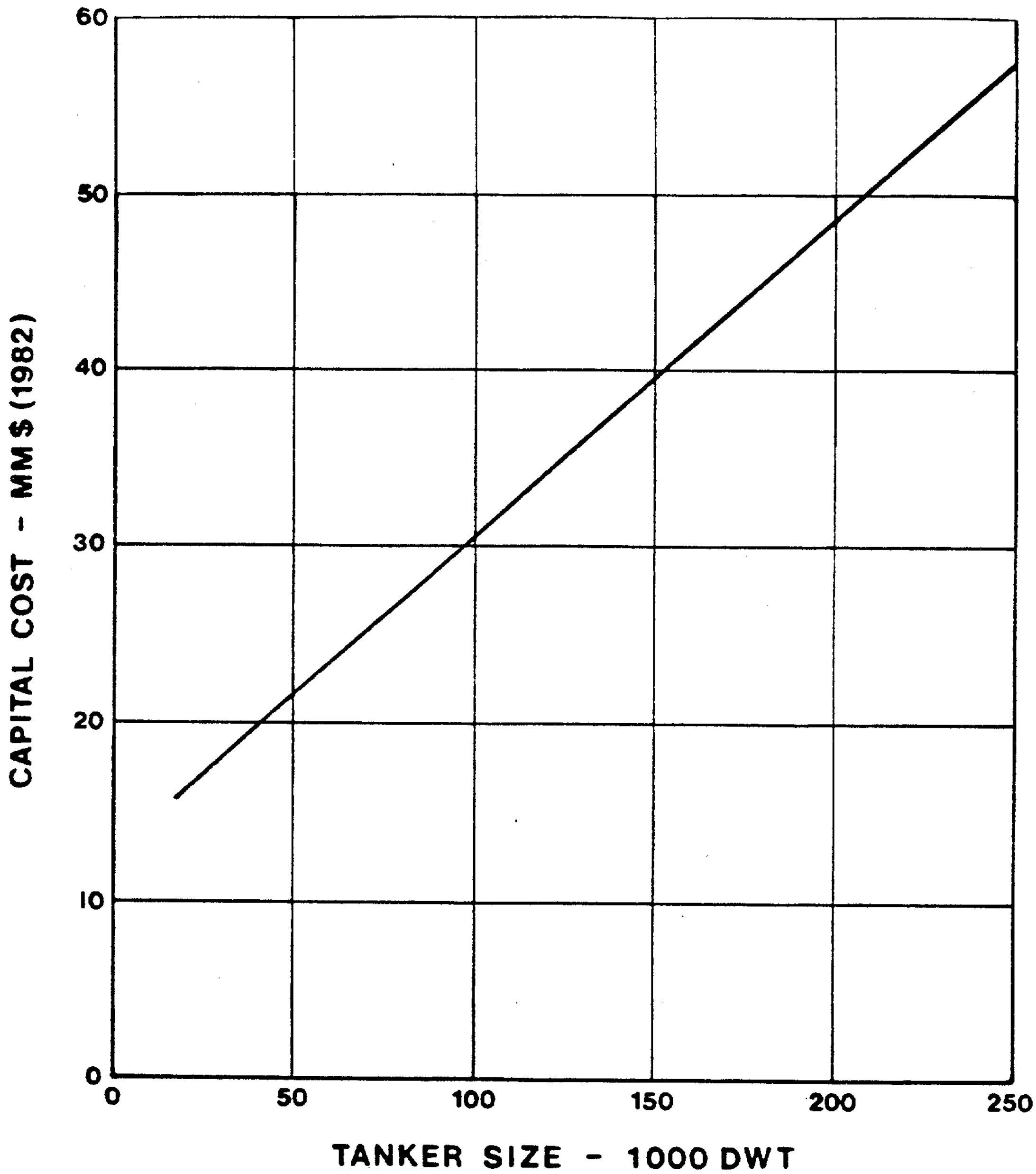


FIGURE 11.2.2 GRAVITY BASED PLATFORM TO SHUTTLE TANKER LOADING PIPELINE CAPITAL COST

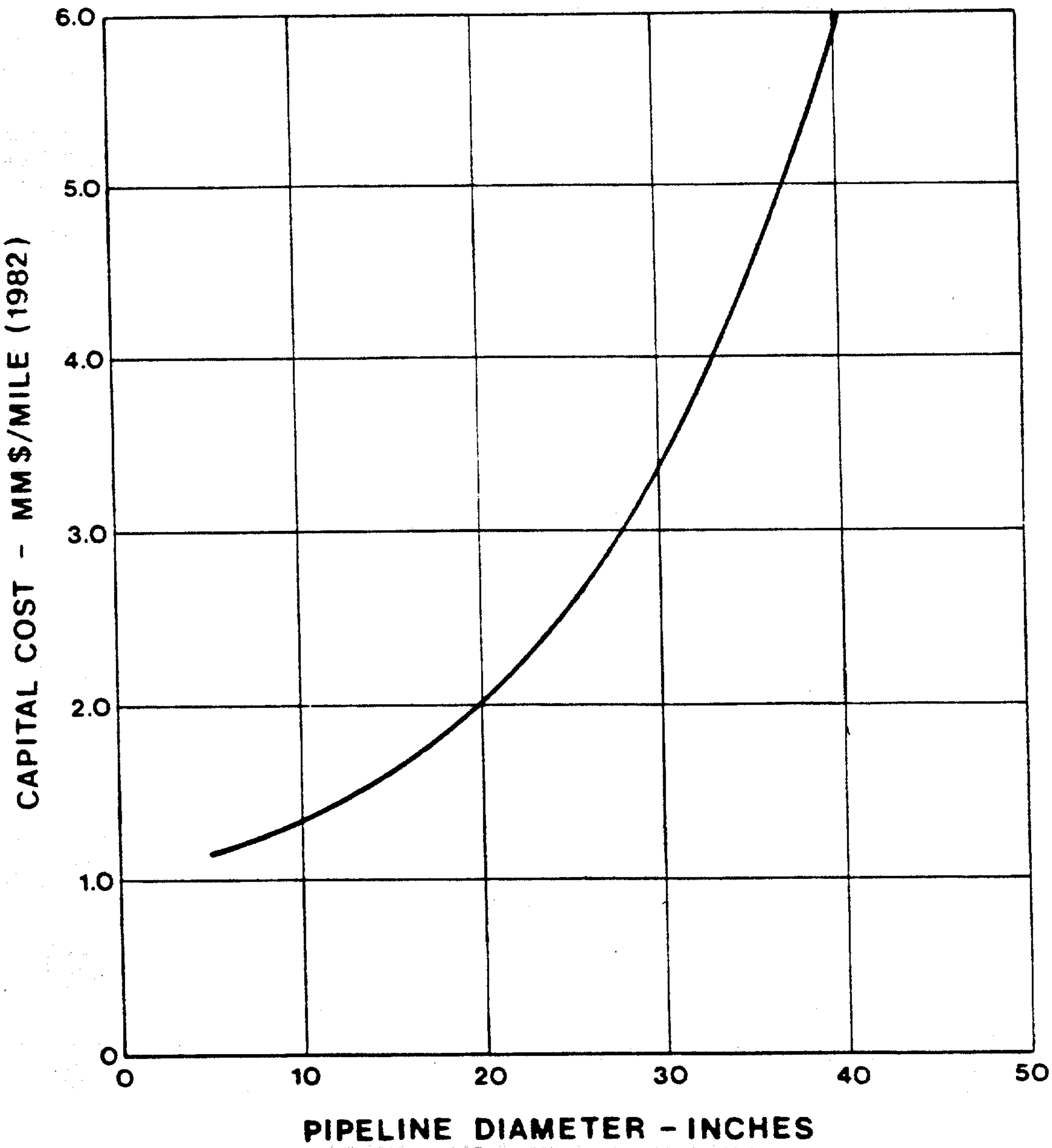


FIGURE 11.2.3 OFFSHORE PIPELINE FROM PRODUCTION PLATFORM TO SHORE CAPITAL COST

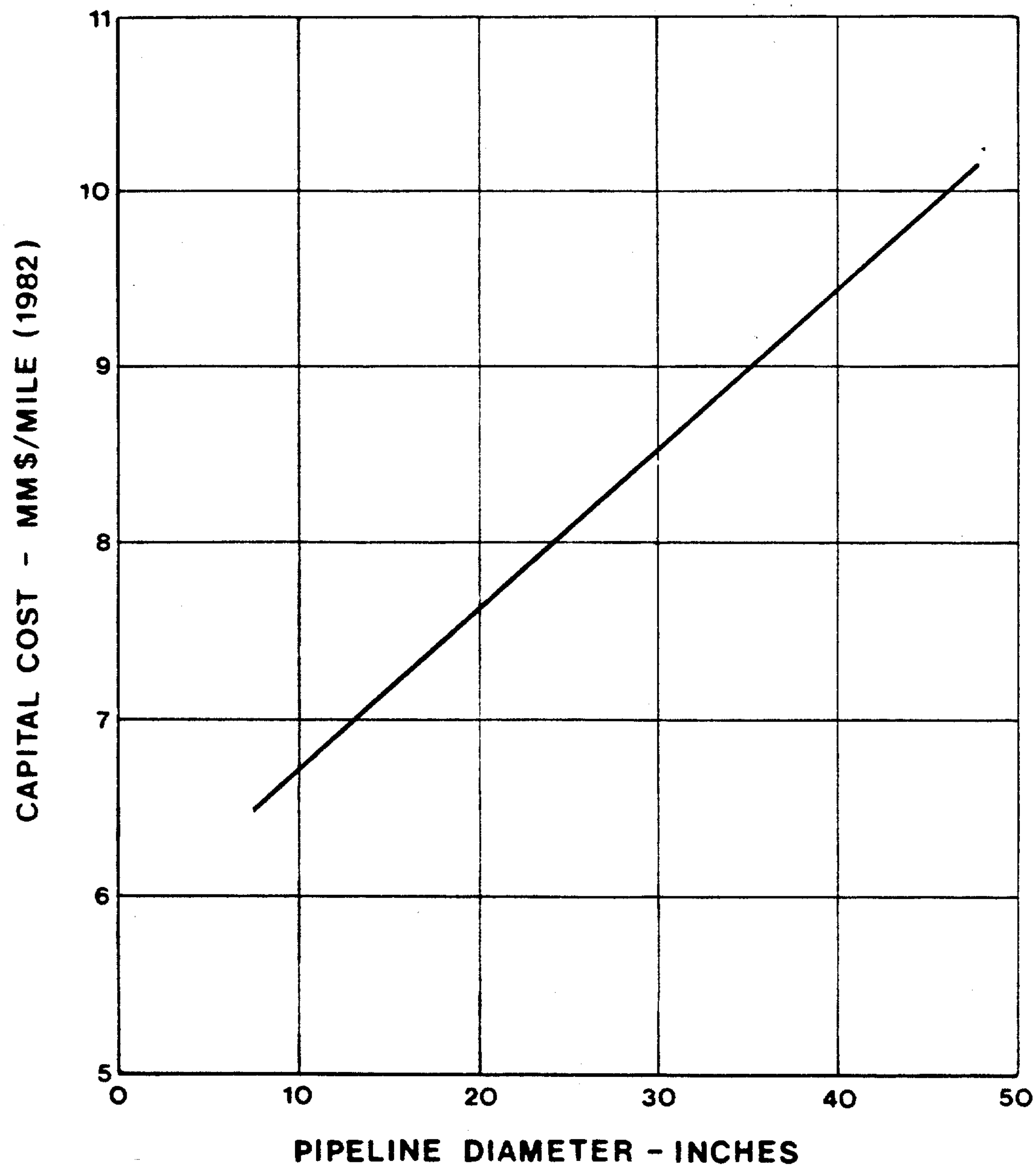


FIGURE 11.2.4 LAND PIPELINE CAPITAL COST VERSUS PIPELINE DIAMETER

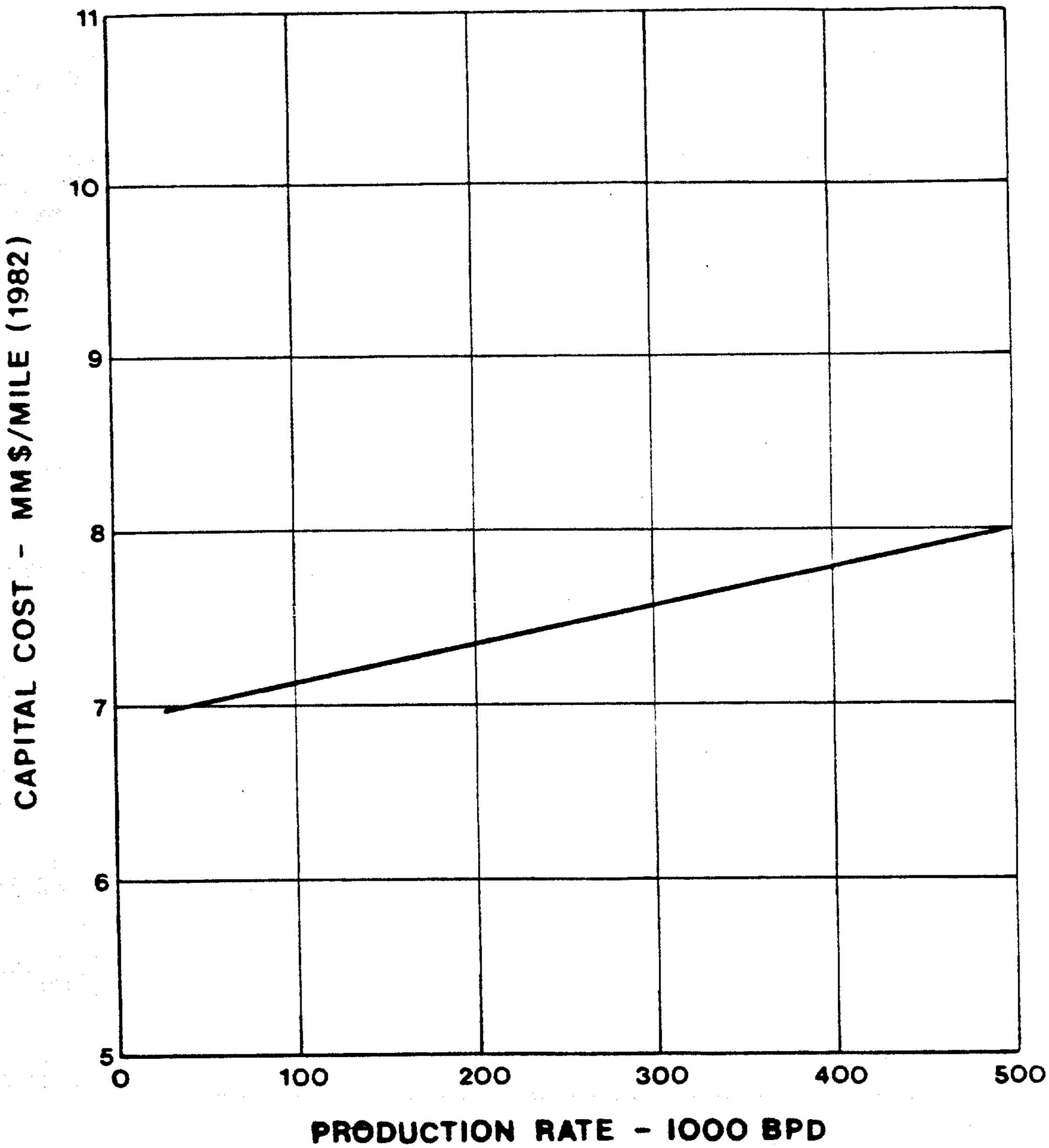
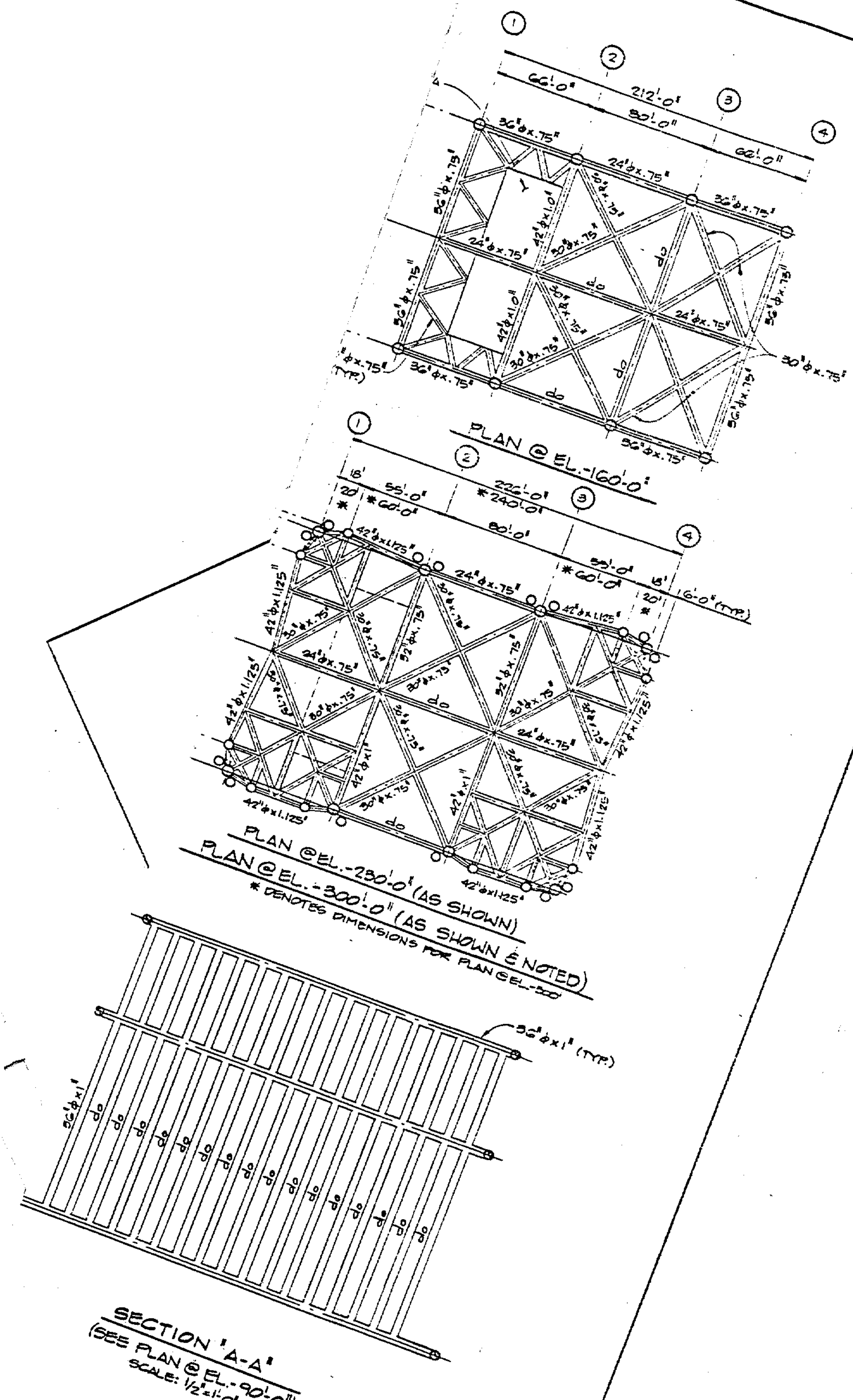


FIGURE 11.2.5 LAND PIPELINE CAPITAL COST VERSUS PRODUCTION RATE



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DESIGN ENGR | DATE

JOINT INDUSTRY PARTICIPANTS

PILED JACKET FOR NORTH ALEUTIAN BASIN

PLANS, ELEVATIONS & SECTION

JACKET FOR 300'-0" WATER DEPTH

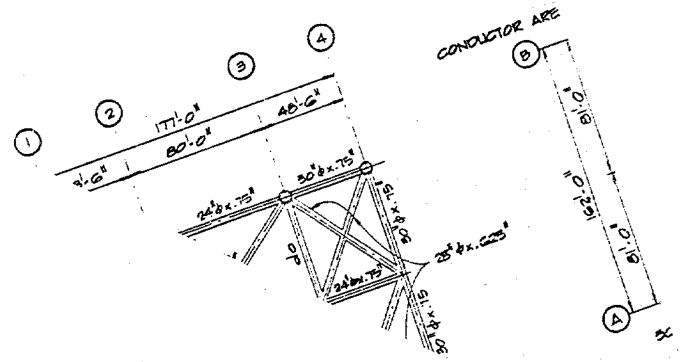
APPROVED BY:

DATE:

PROJECT NO. 284

DRAWING NO.

REV...



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PERMIT APPL.	
BIDDING	
MAT'L ORDER	
FABRICATION	
ERECTION	

SEAL

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10'-0"