PROPRIETARY INFORMATION
UNTIL MAY 1, 1987

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

FINAL REPORT

TABLE OF CONTENTS

Detailed indices are included in each Section.

TABLE OF CONTENTS

SECTION 1.0

Page

LIST OF FIGURES

SECTION 1.0

LIST OF TABLES

SECTION 1.0

TABLE 2

SOUTHERN BERING SEA

PRODUCTION SYSTEMS STUDY

PLATFORM COST SUMMARY

NAVARIN BASIN

(U.S. $ x 10^6)[1,2]

	100,000 BPD	300,000 BPD		
	450'	300'	450'	600'
Steel Structure				
- Installed Structure	313	299	337	403
- Topside and Hookup	549	808	808	808
- Total	862	1107	1145	1212
Concrete Structure[3]				
- Installed Structure	545	542	582	724
- Topside and Hookup	561	809	809	809
- Total	1106	1351	1391	1531

[1]All costs include a 30 percent contingency

[2]June 1982 costs, no escalation

[3]Includes 1.2 million barrels of oil storage. Subtract $100 x 10^6 for no storage case

1.0 DESIGN BASIS ENVIRONMENTAL CONDITIONS

This section documents the background of the design basis environmental conditions utilized in the study of production sytems in the Southern Bering Sea. These design basis conditions should not be confused with design criteria appropriate for final design of production systems for this area. The design basis conditions and the associated ranges in these conditions (identified as lower basis and upper basis) are intended to bracket plausible design conditions. The design basis conditions provide a basis for screening alternative concepts of systems and components, for determining their projected or estimated costs, and for identifying major engineering problems, constraints, and information needed to allow development of production systems to go forward without undue delays or unanticipated costs.

1.1 Introduction

1.1.1 Purpose

The purpose of the investigation of design basis environmental conditions that is summarized in this section was to provide a reasonable basis upon which the alternative concepts for production systems in the Bering Sea could be investigated. Further, so that the feasibility of the most attractive of these concepts could be determined to make reasonable estimates of their costs and associated engineering problems.

The purpose of this particular phase of the study was not to perform an extensive study of background data, nor to perform studies intended to add to the bank of information required to define environmental criteria.

Rather, the purpose was to synthesize information provided by the study participants, provided by the background of the consultants performing this study, and provided by information in the public domain. This information was then used by the consultants and the industry Steering Committee for this project to develop design basis environmental conditions appropriate for this study.

1.1.2 Scope

This study of environmental conditions in the Bering Sea addressed three specific areas. These areas are identified in Figure 1-1 as Navarin Basin (NB), St. George Basin (SGB) and North Aleutian Basin (NAB).

This study addressed four categories of environmental constraints: (1) Soils, (2) Oceanography, (3) Earthquakes, and (4) Ice. These constraints are addressed in the following portions of this Section. The evaluation of the constraints to define the design basis conditions and engineering problems are the topics of the last two portions of this section.

The information background scope of this study was limited to a review of information in the public domain concerning the environmental constraints cited, that in the background of the consultants working on the study, and that provided to the consultants by the participants in this study. Proprietal and confidential information accessed by the consultants or provided by the participants has been integrated into the results of the study. The data or information itself has not been compromised.

One of the most valuable elements of this portion of the study was the interaction between the participants' Steering Committee and the consulting team. This interaction developed through the participant's providing input information to the consulting team, the consulting team reviewing this and other pertinent sources of information, then proposing design basis conditions and associated ranges to the Steering Committee for their review and comment. The result was a collective synthesis of information, interpretations, and judgments into the results documented here.

Two important sources of information were utilized in this study. The first is that provided to the study by the offshore oil and gas industry cooperative efforts in the form of public documents summarizing results of extensive industry-sponsored research and development programs. Most notable were the Alaska Oil & Gas Association "Recommended Research Projects for the Alaskan Bering Sea" (January 1980), and the National Petroleum Council's report, "U. S. Arctic Oil and Gas" (November 1981).

The second important source of information was that provided in the documents resulting from Federal Agency scientific programs. Most notable are those from the U. S. Geological Survey, National Oceanographic Atmospheric Administration, and Cold Regions Research Laboratory of the U. S. Army Corps of Engineers. Essential background on soils, geo-hazards, meteorology, oceanography, seismicity, and ice conditions were provided to this study through the public reports documenting results from these programs (National Research Council, 1980).

1.1.3 Summary of Design Basis Conditions

The approximate locations of the St. George, North Aleutian Shelf and Navarin sedimentary basins are shown in Figure 1-1. These basins are near the southern limit of annual sea ice coverage. As a result, open water prevails in these areas for approximately eight months out of the year, with mid-June to mid-October nearly ice free.

In general, the sea ice environment for the St. George basin and the North Aleutian Shelf basin is less severe than for the Navarin basin. Primary sea ice features in these two basins may include sheet ice, broken ice and first year pressure ridges, not necessarily occurring every year. Scattered drift ice is also characteristic of the south-east St. George and south-west North Aleutian Shelf areas.

In contrast, the Navarin basin has sea ice every year, with multiple rafting of sheet ice producing a total thickness of up to 10 feet. Significant ice loads may also occur in the northern Navarin area from single year unconsolidated pressure ridges. In addition, the ice in the Navarin basin is very dynamic, although the movement rates are not well documented.

Study of the local soil conditions in the south Bering Sea indicates that the surficial soils in the Navarin basin consist primarily of silts (up to 50 percent) and fine sands (25 percent) with little clay. In the North Aleutian basin, the surficial deposits are considerably more sandy (approximately 75 percent sands and 25 percent silts) with the finer sands generally overlaying the coarser sands. In the St. George basin, the surficial deposits range from sands to silts to soft clays, where the fine grained soils are located along the center of the graben that forms the basin. In general, the sediments along the shelf are generally unconsolidated to depths of two to three meters and are loose to soft in consistency.

The seismic activity common to these three sedimentary basins is significantly less severe than that just south of the Aleutian Island chain. St. George and North Aleutian Basins are considerably more active than the Navarin Basin. Furthermore, the seismic activity in the St. George Basin increases going from north to south.

Faulting, with related surface features, is relatively minor in the North Aleutian and Navarin Basins. In contrast, a significant amount of faulting is present in the St. George Basin. Major faults are generally parallel to the long axis of the basin and are located along or on the basin boundaries. These faults often offset the sea floor. In contrast, the minor faults are generally distributed throughout the basin. From the difference in seismicity and the faulting of the different areas, it is apparent that the ground motions resulting from near-field and far-field events will be different in each of the three basin areas.

The meteorologic-oceanographic environment of the Bering Sea is similar to that of the North Sea. Broad, low-pressure storm systems move in a generally easterly direction across the Aleutian Islands. Open water fetches of 2000 and 1000 miles in east-west and north-south directions, respectively, characterize this sea. Ice coverage during the winter storm months may decrease the north-south fetch to 500 miles. Maximum extreme and operational sea states are expected to be similar to the North Sea;

annual and 100-year expected maximum wave heights are in the range of 60 to 90 feet, respectively.

Design guidelines for permanent production systems to be sited in the Bering Sea have been defined by the American Petroleum Institute. Figure 1-2 summarizes API design wave heights, steepness, and deck clearances for this area. For the Bering Sea, a reference level wave height of 85 feet and wave steepness of 1:13 are indicated. Guideline wave heights fall in the range of 75 to 95 feet. The recommended reference level deck clearance is 63 feet above MLLW.

Figure 1-3 summarizes API design earthquake effective horizontal peak ground accelerations and response spectra. For the NAB and SGB, effective ground accelerations of 0.2 g are indicated. For the NB, an effective ground acceleration of 0.05g is indicated.

For shallow and deep alluvium covered locations (typical of these basins), the spectral accelerations in the 1 to 2 second range for NAB and SGB would be 0.24 to 0.12 g for shallow alluvium locations and 0.36 to 0.18 g for deep alluvium locations. Values for NB for the 1 to 2 second range, would be 0.06 to 0.03 g for shallow alluvium locations and 0.09 to 0.05 g for deep alluvium locations.

API has not developed definitive guidelines for ice loadings in this region; however, an API Bulletin has been issued for planning, designing, and constructing fixed offshore structures in ice environments (API, 1981). Ice loading considerations outlined in this document were used in development of the design basis conditions and formulation of design basis environmental loading determination procedures.

Other design guidelines utilized in this formulation of design basis conditions were those issued by the U. S. Department of Interior, USGS, Conservation Division (USGS, 1979) and those issued by the American Concrete Institute (ACI, 1978). Both of these guidelines were utilized to assist in formulation of design basis environmental loading determination procedures, as well as to assist in formulation of the structural and foundation analysis and design procedures.

Tables 1-1 to 1-3 summarize the design basis and sensitivity range environmental condition parameters for this study of Bering Sea Production systems for each of the three basins. The "Design Basis" conditions were those utilized for the "Base Case" studies of production system concepts and those utilized for the similar studies of the feasibility of the two most attractive systems.

The "Sensitivity Range" parameters are those investigated to determine the influence on the costs and engineering aspects of the two most attractive production systems chosen for the feasibility study portion of this work.

These sensitivity range parameters generally were treated as independent parameters in determining cost and engineering influences on the production systems chosen for the feasibility study.

Key design basis parameters for each of the three basins are summarized as follows (refer to Figure 1-1):

Basin	Water Depth ft	Wave Height ft	Ground Acceleration g	Ice Loading on Proj. Area kips/ft	Soil Types
NB	450	80	0.05	115	clay, sand
SGB	450	85	0.125	85	clay, sand
NAB	300	78	0.24	85	sand

These design basis parameters are in good agreement with those suggested by API (API 1981) and the USGS (USGS, 1979).

Figure 1.4 defines the types of surface soils anticipated in each of the three basins. Figures 1-5 and 1-6 characterize the soil shear strength and soil effective angle of internal friction for cohesive and granular soil profiles, respectively, anticipated for these locations.

Figures 1-7 to 1-9 define the acceleration response spectra for SGB, NB, and NAS, respectively, as scaled with the effective horizontal ground accelerations indicated in Tables 1-1 to 1-3, and recognizing the influence of anticipated local soil conditions.

1.1.4 Summary of Major Engineering Information Needed

This section summarizes major engineering information needed pertaining to environmental conditions and criteria to allow development of the Bering Sea Basins to go forward without undue and significant delays, costs or risks.

This evaluation has been based on the information which has been cited in this study and in the subsequent findings of the studies of production system concepts. Additional background on many of the engineering efforts identified here can be found in the report by the Alaska Oil & Gas Association, "Recommended Research Projects for the Alaskan Bering Sea" (AOGA, January 1980).

Table 1-4 identifies 22 specific efforts intended to provide information for design and siting of production systems in the Southern Bering Sea. These efforts have been organized into five categories: (1) Soils, (2) Oceanography, (3) Earthquake, (4) Ice, and (5) Design Criteria. Justifications or applications for each of the efforts are identified in Table 1-4.

It is important to recognize that the industry presently has a very intensive program of arctic research underway. This and the complementary work by government amd academic organizations, some of which is specific to the Southern Bering Sea, will provide elements of the efforts which have been defined in Table 1-4. In addition, the study team has been privileged to review and participate in only a small part of the past and current proprietal and nonproprietal work by industry, government, and academic organizations. Thus, efforts or components of efforts may have been identified in Table 1-4 which have already been undertaken or are in the process of being undertaken.

Summarizing the efforts identified in Table 1-4:

- Soils: Site specific soil borings, route specific soil surveys, geohazard and bathymetric surveys and studies for platform sites; geohazard and bathymetric surveys and studies for pipeline routes; scour potential evaluation.

- Oceanography: Wave measurements, current measurements, wind measurements, design conditions, operating conditions.

- Earthquakes: Local geologic and site effects; seismic source and seismic exposure evaluations.

- Ice: Ice measurements, ice features, design conditions, operating conditions, wave-ice interactions, ice-flexible multi-leg structure interactions.

1.1.5 Acknowledgments

The project Steering Committee participants involved in the study of environmental conditions were as follows:

- Mobil - George Vance
- Chevron - Enrique Gartenbaum
- Amoco - Dan Gibson
- Gulf - Brian Thomas
- Arco - Jerry Machemehl

The project consulting team involved in the study of environmental conditions was as follows:

- Soils - J. P. Singh, Harding-Lawson Associates
- Earthquakes - J. P. Singh, Harding-Lawson Associates
- Oceanography - R. G. Bea, PMB Systems Engineering Inc.
- Ice - K. Vaudrey, Vaudrey and Associates; R. G. Bea, PMB Systems Engineering Inc.

Appreciation is expressed to the individuals cited for their contributions to this study. Special acknowledgment is due to Mobil Research and Development Corporation for their assistance in performing the wave-ice interaction studies that were a part of this effort.

1.2 Soils

Surveys of the distribution of different sediment types have been completed for much of the Bering Sea. In the St. George Basin, the U. S. Geological Survey has undertaken a program to determine the geotechnical properties of the sediments in the nearshore waters. Similar work is also planned for other areas. As customary, detailed site-specific geotechnical-geophysical surveys will be an integral part of exploration and production activities.

An overview of the soil distribution is given in Figure 1-4. In general, the St. George, North Aleutian, and Navarin Basins are sediment-filled structural basins within the continental shelf. The distribution of the soils within the region, their origin and properties as well as potential geological hazards are discussed in the sections which follow.

1.2.1 Bathymetry

The Bering Sea may be characterized as having a gently sloped offshore continental shelf. Generally, water depths increase gradually 2 to 3 feet per mile in many areas. No unusual bathymetric trends are believed to be present in the shelf area. General bathymetry is well known with numerous navigation charts and other bathymetric maps available. Development of more detailed bathymetry data may be desirable prior to conducting operations in a given area.

The general bottom features of the region are given in Figures 1-10 and 1-11.

1.2.2 Sediment Distribution

The distribution of sediments within the Bering Sea is a direct result of the geological history of the area. Broadly speaking, data collected on the inner shelf of the Bering Sea indicates that surface sediments consist largely of granular materials decreasing in grain size away from the coast. Limited coring data indicates grain size increases with the depth below the sea bottom. Outer shelf data indicates that sands still predominate. However, in deeper water, clayey silts make up more than half the sediments.

In the succeeding sections, more detailed information is given for each basin.

.2.1 St. George Basin

Description of Site

As shown in Figure 1-10, the St. George basin area, located in the southern Bering Sea, can be subdivided into four major physiographic provinces: outer continental shelf, continental margin, Pribilof ridge and the Bering and Pribilof canyons. The outer continental shelf is a broad, flat area that has a gradient of 1:20,000 (0.03 degrees) between the 300-ft. isobath and the shelf break at about 550 ft. The Pribilof ridge is a northwest-southeast-trending topographic high that is capped by the Pribilof Islands. The ridge is a relatively smooth surface cut by at least one terrace that may be a Pleistocene feature. Toward the southeast, the ridge plunges below the shelf southeast of St. George Island but can be followed to the southeast in the subsurface for an additional

125 miles. On the northwest, the continental slope descends abruptly from the shelf break with gradients of 1:20 (3 degrees). Toward the southeast, the gradient is flatter, averaging 1:40 (1.4 degrees). The continental slope is characterized by hummocky topography, scarps, and canyons. The continental margin in this region is incised by two large submarine canyons, Bering and Pribilof (Scholl, et al., 1969), which may have played significant roles in the westward transport of sediment into the Aleutian basin.

Present-day sediment dynamics, combined with the dynamics associated with lowering of sea level during the Pleistocene, have created a mixture of sediments on the outer continental shelf of the southern Bering Sea which has been derived from the Alaskan mainland, the Aleutian Islands, and the Pribilof ridge. Concentrations of fine-grained, highly organic sediments in the St. George basin represent a further variation in the regional sediment composition patterns. A distribution of surface sediment size over the Southeastern Bering shelf is given in Figure 1-12.

Lack of present-day currents sufficient to move even clay-size material and the presence of the Bering submarine canyon between the Aleutian Islands and the outer continental shelf and slope indicate that Holocene sediment dynamics cannot be used to explain the observed distribution of surface sediment derived from the Aleutian Islands. The sediment distribution pattern is believed to be relict and a result of sediment dynamics which existed at lower sea levels during the Pleistocene.

Soils

The lease sales area overplotted on the bottom sediment map developed by Gardner, et al., 1979 (see Figure 1-4) shows that the bottom sediments vary in soil type from sand to silty sand to sandy silt to silt to silty clay.

The assumed engineering properties of the soils are discussed in Section 1.2.3.1.

1.2.2.2 North Aleutian Basin

Description of Site

The North Aleutian basin lies within the continental shelf immediately north of Unimak Island and Alaska Peninsula (see Figure 1-4). Bathymetric contours indicate water depths in the range of 150 to 350 meters with flat gradients on the north and a moderately steep rise toward the southeast against the Unimak Island and the peninsula.

Soils

The lease sales area overplotted on the bottom sediment distribution map (see Figure 1-4) shows that the bottom sediments are primarily sandy in nature. In addition, the distribution of surface sediment size over the southeastern Bering Sea shelf including the North Aleutian basin is given in Figure 1-12. In general, the sands become finer with increasing distance from the shoreline.

1.2.2.3 Navarin Basin

Description of Site

There are three physiographic provinces in the Navarin basin area (see Figure 1-10). There are (1) the flat, wide, continental shelf (2) the steep, rugged continental slope, and (3) the rise. Three large submarine canyons deeply dissect the outer shelf and slope.

Shelf

The 70,000 mi^2 of Bering shelf that includes the Navarin Basin lie between 300-ft. and 500-ft. isobaths (see Figure 1-4). The width ranges from about 75 miles at the northern and southern parts of the study area to a maximum width of about 145 miles in the central part. The average gradient is about 0.02 degrees as compared to world-wide average of 0.12 degrees for continental shelves reported by Shepard (1963).

Slope

The continental slope forming the southeastern boundary of the Navarin basin province begins at the 500-ft. isobath and extends to a depth of 9200 feet northwest of Zhemchug Canyon (Figure 1-10). Southeast of this canyon the slope abruptly changes gradient at 7800 feet. The slope includes an area of about 18,000 mi^2. The gradients of the Navarin slope range from 3 degrees to 8 degrees with even steeper gradients over shorter segments of slope. For comparison, the world-wide average gradient for continental slopes is about 4.3 degrees (Shepard, 1963). The width of the continental slope ranges from 30 miles in the middle of the province to 12 miles south of Zhemchug Canyon. Exceptions to these width and gradient values are within the three submarine canyons that are deeply incised in the shelf.

The three major submarine canyons that cut deeply into the Bering continental margin (Figure 1-10) are Navrinsky, Pervenets, and Zhemchug. All three canyons head in water depths less than 500 ft. Extensive deep-sea fans have been built at the mouths of the canyons in water depths of about 10000 ft. Navarinsky is the longest canyon (210 mi), Pervenets the shortest (80 mi) and Zhemchug intermediate in length (150 mi). Both Navarinsky and Zhemchug Canyons are about 62 mi. wide at the shelf break, but the smaller Pervenets Canyon is only 19 mi. wide. Wall relief of the three canyons at the shelf break ranges from 2300 ft. for Navarin and 260 ft. for Pervenets, to 8500 ft. for Zhemchug.

These canyons are incised into Neogene and older, more lithified Paleogene rocks, principally mudstones, that are thought to make up much of Navarin basin (Marlow et al., 1976). The canyons, especially Zhemchug, are apparently structurally controlled, the structures dating back at least into the Paleogene (Scholl et al., 1975). The major cutting of the canyons probably occurred when glacio-eustatically lowered sea levels exposed most of the Bering shelf.

Rise

The continental rise in this remote area has been sparsely sounded, but is a prominent enough feature to be easily recognized on profiles that traverse the bounding features--the Bering continental slope and the

Aleutian abyssal plain. The rise includes 15,000 mi^2 of the Navarin Basin province and begins at the base of the slope. Depths at the base are 9200 ft. northwest of Zhemchug Canyon and 7900 ft. south of the canyon. Depths extend to the 12000 ft. isobath which marks the beginning of the abyssal plain. The width of the rise averages about 50 mi, ranging from a minimum of 16 mi northwest of Zhemchug Canyon to more than 60 mi adjacent to the mouths of the three large canyons. The gradients across the rise range from a low of 0.5 degrees adjacent to the canyon mouths to 1.8 degrees off the two prominent ridges located northwest of Zhemchug Canyon.

Deep-sea channels cross the rise in the area of the canyon mouths and are apparently connected to the submarine canyons. Gravity cores collected near the mouths of the canyons and on the adjacent rise contain sand lenses that indicate deposition by turbidity currents (see Carlson and Karl, 1981). Seismic reflection profiles of the rise also indicate that presence of turbidities which, together with the cores, suggest that the rise, at least adjacent to the canyons, consists of deep-sea fan deposits. The gradients measured across the rise near the canyons are also similar to gradients reported from other deep-sea fans.

1.2.3 Engineering Properties of Soils

The soil properties of the basins were obtained from drop core samples of the bottom, plus generalized data from four deep borings for the St. George basin. Because of the lack of deep borings for the Navarin, North Aleutian basin, soil properties were assumed to be like those of the St. George Basin due to similarities in the deposition of the materials in the two areas. The exception to this is the presumed absence of cohesive deposits in the North Aleutian Basin.

1.2.3.1 Permafrost

No observations or predictions of offshore permafrost have been reported for the Bering Sea. The absence of permafrost in the numerous shallow cores taken in various areas indicates a lack of shallow permafrost horizons. Although there is less deep coring data, there have been no observations of permafrost in the available core logs. Therefore at this time, deep permafrost zones are not anticipated.

1.2.3.2 St. George Basin

In order to develop soil strength parameters for design, the soil types were classified into two basic categories, i.e., cohesive and granular. The soil information used to evaluate the strength characteristics was obtained from: (1) bottom sediment information by Gardner et al., 1979; (2) the cost well drilled in the middle of the basin during 1977; and (3) three proprietary borings drilled during 1981.

Based on this information, the strength profiles for granular soils are shown in Figure 1-6, and for clayey soils in Figure 1-5. The variations in strength profile of sandy soils are intended to reflect the variations in the density and grain size characteristics within the profile. On the other hand, the shape of strength profiles for clayey soils are intended to reflect a crust-like nature. Due to variations in factors forming this crust, the maximum strength and its depth in the three profiles has been varied in order to get an estimate of the bounding conditions.

1.2.3.3 North Aleutian Basin

The lease sales area overplotted on the bottom sediment distribution map (see Figure 1-4) shows that the bottom sediments are primarily sandy in nature. In general, the sands become finer with increasing distance from the shoreline. Information on strength characteristics of the soils from this basin is non-existent. Because of the similarity in deposition of the sandy materials in St. George Basin, strength profiles developed for sandy soils in St. George Basin (see Figure 2-6) were also used for the North Aleutian Basin.

1.2.3.4 Navarin Basin

The bottom sediments in Navarin Basin vary from sands to clays (see Figure 1.4). Because the location of lease sale area with respect to the basin is not known, the range of soil properties developed here are based on the 68 gravity cores taken in the Navarin Basin by the USGS (reference). The preliminary consolidation data indicates that the shelf sediments are lightly to heavily overconsolidated.

Because no strength data are available for the sandy soils, the moisture content information obtained from the gravity cores was used to estimate strength profiles for granular soils. Because of the similarity in deposition of the sandy materials in St. George Basin, strength profiles developed for sandy soils in St. George Basin were also used for the Navarin Basin. On clayey soils, the strength information pertains only to depth within 15 feet of the mudline. Due to similarities with St. George Basin clayey soils, the strength profiles developed for St. George Basin were also adopted for the Navarin Basin. These strength profiles are shown in Figures 1-6 and 1-5, respectively.

1.2.4 Geohazards

Three sea floor geological hazards are discussed, basin by basin, in the following subsection. A final sea floor hazard, that of earthquake faulting, is discussed in Section 1.4. Specific site locations of geohazards should be pinpointed and their extent evaluated by a survey program and soil sampling where exploration or production operations are concerned.

1.2.4.1 St. George Basin

Liquefaction

The nature of sea floor sediments indicates that the soils are susceptible to liquefaction. In shallow water depths, liquefaction is likely to occur from both seismic and storm wave induced loading conditions. In deep water depths, liquefaction is likely to be more severe for seismic loading conditions, with a decrease in potential towards the northwest due to decrease in seismicity (see Figure 1-13).

Slope Stability

The regimes of St. George Basin that show unstable sediments (e.g., gravity slides, slumps, creeps, scarps, etc.) are confined to the continental slope and rise and two major submarine canyons (Pribilof and Bering). These zones of creep begin near shelf break at depths of about 550 ft and continue on to the upper continental shelf. Furthermore, hummocky topography occurs on the continental slope on a large scale and mass movment is a common feature.

Gas-Charged Sediments

Anomalous concentrations of gas have been measured in surface and near-surface sediments; they appear to be associated with boundary fault (major faults delineating St. George Basin and the north side of Pribilof Ridge).

None of these sea floor hazards have been considered in the feasibility studies. However, the following problems related with the above-mentioend sea floor hazards should be considered during the site-specific studies:

1. Blowout hazards along boundaries of the basin.
2. Liquefaction and submarine slide problems along continental slopes and major canyons.
3. Mass sediment movement and focusing of currents in two submarine canyons.
4. Future movements along faults that could disrupt structures crossing these faults.

1.2.4.2 North Aleutian Basin

Liquefaction

Due to the sandy nature of bottom sediments in the North Aleutian Basin, liquefaction is likely to occur from both seismic and storm-induced loading conditions.

Slope Stability

The bottom in North Aleutian Basin is very smooth, with slopes less than 2.5 percent. Sediment movement in this basin is related to strong currents and scouring.

None of these sea floor hazards has been considered in the feasibility studies. However, the following problems related to the above-mentioned sea floor hazards should be considered during the site-specific studies:

1. Active scouring
2. Liquefaction

1.2.4.3 Navarin Basin

Liquefaction

The potential for liquefaction in Navarin Basin is low due to deep water depths and low seismicity.

Slope Stability

There is no slumping on the shelf. However, there is slumping on the walls of canyons along continental slopes. In addition, there are sand waves in heads of canyons.

Gas-Charged Sediments

There are isolated small to large areas of gas-charged sediments on the shelf.

None of these sea floor hazards has been considered in the feasibility studies. However, the following problems related to the above-mentioned sea floor hazards should be considered during the site-specific studies:

1. Possible isolated areas of blowout hazards.
2. Submarine slide problems along the continental slope

1.3 Oceanography

A knowledge of both the normal and extreme oceanographic and meteorological conditions is important in assessing sites in the Bering Sea. Normal conditions must be known for selecting equipment and scheduling exploration, construction, and production operations, while extreme conditions must be known for the design of the structures and pipelines to be installed.

1.3.1 Meteorology

The meteorological descriptions given in this section are general in nature due to the wide variation within the Bering Sea. The meteorological background for this area has been derived primarily from coastal meteorological stations and observations from ships passing through the area. References pertaining to the local conditions at specific sites can be found in the Alaska Oil and Gas Association (AOGA, 1980) much of the information from that publication is summarized here.

The weather over the Bering Sea is very changeable. Wind shifts are frequent and rapid. The late spring and summer seasons have considerable fog and rain. In early fall, gale frequency increases, the occurrence of fog lessens, and snow is likely any time after mid-September. Storminess increases in late fall and early winter. Heavy winds from any direction are usually accompanied by precipitation; however, the rain or snow that occurs with easterly or southerly winds is likely to continue steadily until the wind shifts, while rain or snow squalls are characteristic of the westerly and northerly winds.

There is considerable cloudiness over the Bering Sea with an average of from 60 to 80 percent of the sky covered the year round. There is usually, however, considerably more cloudiness in mid-summer than during the winter months.

The normal storm tracks, Figure 1-14, in the Bering Sea move from the North Pacific eastward along the Aleutian chain and northward through the Bering Strait or across the Alaskan coast. Occasionally, storms will develop within the Bering Sea and move north or northeastward, creating strong winds along the western coastal area. Major storm tracks in the Bering Sea are shown in Figure 1-15 during the period 1949 to 1975.

The barometric activity of the North Pacific is dominated by the intense Aleutian Low Pressure from October through March. The pressure in this Low (whose winter center lies over or near the western Aleutians) usually begins to fall in August, continues to fall until December or early January, and rises in spring to a peak in June or July. The strongest pressure gradients are in the stormy winter. Most of the traveling cyclic disturbances pass through the Aleutian Low in west-to-east or south-to-north movements. These storms frequently cover a large area and may produce strong winds all along the Aleutian chain. A synoptic map of a typical low pressure storm system in the Bering Sea is given in Figure 1-16.

Fog is reported in 11 to 25 percent of the year-round observations. Very low visibility (less than 1/2 mile) is reported about one-tenth of the time and very good visibility (greater than 10 miles) is reported over one-third of the time. The visibility is generally poorest in the summer and best in the autumn. Summer fogs generally do not extend higher than 100 feet above the water surface.

The offshore air temperature over the Bering Sea generally decreases with increasing latitude. For example. the annual mean air temperature at 55°N offshore is 42°F and at 65°N is 34°F. The mean monthly air temperature offshore ranges from a February low of 35°F to an August high of 51°F in the southern Bering Sea, and from a low of 5°F to a high of 48°F in the northern Bering Sea. From 25 years of ship observations, the highest air temperature was 71°F and the lowest was -22°F. At the mainland Alaska coastal locations there are 200 to 250 days per year of temperatures less than 32°F and 60 to 100 days per year below 0°F.

Over the entire Bering Sea precipitation in some form was recorded in an average of 22 percent of all observations, the greatest precipitation occurs from January through April and the least occurs in June and July. The yearly precipitation reported at coastal stations ranges from 35 to 66 inches in the Aleutian Islands and 13 to 20 inches at other locations. One-third to one-sixth of the coastal precipitation is in the form of snow or sleet.

1.3.2 Wind

The annual average wind speed varies from 13 to 16 knots with generally a lower wind speed near the coast (excepting local effects). Gale force winds (greater than or equal to 34 kt) occur an average of 2 to 6 percent of the time year round, with a maximum of about 15 percent in the winter months. The winds are predominantly northerly to northeasterly from

October through May and southwest to southeast from June through September (AOGA, 1980). A characterization of extreme wind speeds for various return periods as well as normal wind speeds are shown in Figure 1-17 (AOGA, 1980).

1.3.3 Waves

As a result of the varying storm tracks previously discussed, open water sites can be subjected to wave attack from nearly all directions. Furthermore, as a storm moves past the site, it creates a confused sea with waves of various periods arriving concurrently from several directions. Normal waves in the Bering Sea are predominantly locally generated and multi-directional, although ice cover during the winter months limits wave generation and propagation from the north. Based on ship observations, the monthly mean wave heights range from three feet in July to 6 to 9 feet during the period October through March (AOGA, 1980).

A proprietary extreme wave study, (Ocean Science and Engineering, 1970) from Bristol Bay and the southern Bering Sea was conducted in 1970. The study used the Wilson wave model to hindcast severe historical storms and extrapolated the results to extreme return periods. Extreme wave heights for platform design criteria have not been defined adequately throughout the Bering Sea. An estimate for extreme wave height and steepness for the Bristol Bay area was presented in the 1979 revision of API RP 2A. This guideline value was based on available information at the time and should be considered subject to revision (AOGA, 1980).

A plot of the range of expected maximum wave heights in the Southern Bering Sea Basin is given in Figure 1-18 (AOGA, 1980).

1.3.4 Currents

In general, information on summer currents is poor, and for winter currents is nearly nonexistent. It is nevertheless critical that the currents be known because they are the major factor controlling the velocity at which the ice pack drifts (National Petroleum Council Committee, 1981). Much of the important information which is known is cited in the following paragraphs (AOGA, 1980).

While many details of the Bering Sea circulation are unknown at present, certain large scale features have been commonly accepted. These patterns are shown in Figures 1-19 and 1-20. The concepts of inflow of water through the Aleutian Commander Island arc, general northerly flow, the existence of large and small scale gyres within a counter-clockwise circulation and the fact that this is a region of water mass formation, have developed through the work of many investigators, both foreign (Russian, Japanese, etc.) and the U. S. (Univ. of Washington, National Marine Fisheries Service, etc.).

The Alaskan Stream, flowing westward of the south side of the Aleutian Chain, provides inflowing water through the numerous channels of the Aleutian-Commander Island arc chain. The amount of water transported has not been established but it has been accepted that the water exchange can occur at all levels with reversals of flow common. Since the amount of water exchange depends upon the size of the channel, the deeper channels (Kamchatka, Commander-Near and Amchitka) will have the larger flow contributions. Winter intensification of this flow due to the additional wind stress has been postulated but not convincingly proven.

The inflowing water, generally warmer and more dilute than that of the Bering Sea, is soon transformed into subarctic water. In summer, complicated vertical distributions exist because of influence upon the surface layer by precipitation and solar heating. In winter, the water is strongly mixed by the cooling and sinking of surface waters and the passage of atmospheric disturbances.

Water flowing through the Amchitka Pass continues to flow northward, until it is pushed northwestward by the continental shelf. This flow soon reaches the USSR and breaks into north and south coastal currents. Water flowing through the Commander-Near Pass flows northward and loops in a counterclockwise direction to join the southerly coastal current off the USSR. This water continues flowing to the south and leaves the Bering Sea through the Kamchatka Pass. The numerous gyres that exist throughout the Bering Sea further complicate the flow.

Flow through Amchitka Pass also loops clockwise and flows along the north side of the Aleutian Islands toward Bristol Bay. This flow continues along the Alaskan coast toward the north as a very broad and sluggish current. St. Matthew Island breaks this flow into two bands, one following the Alaskan coast northward past Norton Sound and through Bering Strait, while the second flows westward to the USSR coast where it is forced northward and also through Bering Strait.

The newly issued Climatic Atlas of the Outer Continental Shelf Waters and Coastal Regions fo Alaska confirms the general circulation features described above for the summer. Currents, however, are generally less than 1 knot, except in the vicinity of the Bering Strait where they reach a maximum of 2.4 knots. In waters shallower than 150 feet, currents at most locations will be fraction of a knot (National Petroleum Council Committee, 1981).

1.3.5 Tides and Surge

Little reliable, quantitative data on storm surges exist. Storm surges on the north coast of the Alaska Peninsula are rare, and surges on the coastline north of Bristol Bay are limited to the ice-free seasons. The significant surges generally are caused by storms from the southwest.

Tidal ranges vary considerably along the Bering Sea coast as a result of the irregular coastline. Diurnal ranges vary from less than 5 feet along the open coast and in Norton Sound, to 12 feet in Kuskokwin Bay and more than 20 feet at the head of Bristol Bay (AOGA, 1980). At St. Matthew's Island, the tidal range is 3.5 feet.

Tsunamis are reported to be uncommon in the Bering Sea. Because of the large continental shelf, tsunamis are probably dissipated before reaching the coastline. Only two tsunamis have been reported at Port Heiden--one in 1928 (about 6 feet) and one in 1964 (about 4 feet). However, some occurrences may have been unreported because of the sparse population.

1.4 Earthquakes

The seismicity of the St. George, North Aleutian, and Navarin Basins varies appreciably and is, in most part, a function of distance from the northward dipping underthrust junction between the Pacific and North American plates which reaches the sea floor along the Aleutian trench. Seismic activity along this junction (Benioff zone) extends at increasing depth northward beneath the Aleutian volcanic arc (see Figure 1-21) and then terminates at depth in a steepened zone at about the northward limits of the arc. Further north into the Bering Sea, seismic activity diminishes appreciably; no magnitude 5 or greater events have been recorded beyond about 60 mi from the volcanic arc.

Large dextral slip faults which extend in a broad arc through southern Alaska apparently have been relatively quiescent in their more westerly extent in southwest Alaska. Although they cannot be considered inactive based on available data, their earthquake potential, both in terms of magnitude and recurrence interval, is generally agreed to be much less than that of the Aleutian underthrust zone.

Geophysical evidence indicates extensive faulting within the St. George Basin but a paucity of data on earthquakes which might be associated with this faulting makes it difficult to assess the seismogenic potential for these faults. Some works conclude that these faults are non-tectonic "growth faults" incapable of generating earthquakes.

1.4.1 Sources

1.4.1.1 St. George Basin

The St. George Basin is a major structural feature and is basically a graben filled with up to 6 mi of sediment (see Figure 1-22). It trends southeasterly from the Pribilof Islands towards the Alaska Peninsula. The conditions which led to the evaluation of this structure are attributed to former continental margin convergence and underthrusting zone in the Bering Sea (Scholl et al., 1975a). The continental margin approximately coincides with the continental slope southwest of the St. George Basin. Underthrusting at this now abandoned continental margin ceased during late Mesozoic time and has shifted to the modern Aleutian trench. During the Mesozoic and early Tertiary, inner and outer magmatic zones developed in conjunction with underthrusting at the plate boundary (Marlow et al., 1976a,b). The outer zone is represented by the volcanic Pribilof Islands and the inner zone by a broad band of volcanics curving from the Yukon delta area northwest through St. Matthew and St. Lawrence Islands (see Figure 1-10).

Regional isostatic uplift or tectonic rebound of the entire Bering Sea shelf accompanied the cessation of underthrusting at the continental shelf (Marlow et al., 1976a,b). Erosion and regional subsidence and extensional rifting dropped the shelf area below sea level in the Tertiary. Formation of elongate basins along the outer continental margin, including the St. George Basin, accompanied the extensional collapse of the area; the basins contemporaneously filled with Cenozoic sediments as they formed (Marlow et al., 1976a,b and 1977). According to Gardner and Vallier (1977) and Marlow et al., (1977); this extensional rifting at the shelf edge has continued to the present time, as evidenced by recent displacements of the sea floor at the edge of St. George Basin and along the Pribilof ridge.

Faulting has been identified throughout much of the St. George Basin area, but the major faults are concentrated along the edges of the basin (i.e., graben boundary) and also Pribilof ridge (see Figure 1-23). These faults have been interpreted as growth faults by one investigation (Marlow et al., 1977); however, the evidence is not clear. Surface displacement on faults has been observed locally along the basin margins. The distribution of the faults and geologic evidence suggests that the processes which formed the graben are still active (Gardner and Vallier, 1977; Hopkins, 1975) and may be capable of producing earthquakes. The magnitude of the largest event assigned to these faults is 5.9 (OASES); on the other hand, the possibility of magnitude 7.2 in the St. George Basin has also been postulated.

Faults found on St. George Island displace older lava flows by 200 ft, and they displace younger lava flows of less than 100,000 years but older than 15,000 years by about 25 ft, indicating that the faults have been active over a long duration and are probably periodic. Faults in the St. Paul rift between St. Paul and the Otter Islands displace the sea bottom by 13 to 25 feet. The age of the faulting here is not known. The St. Paul rift is on trend with the southern boundary of the St. George basin and is representative of the larger extensional structures of the area.

Although the "growth fault" nature of many of the faults is recognized, it is believed that the regional and local tectonics may still be active

(Hopkins, 1975; Gardner and Vallier, 1977). In the strictest sense, a "growth fault" is not of tectonic origin. It results when lateral constraints do not exist for a prism of material, or where hydrostatic pressures within the strata are reduced during diagenesis, causing the propagation of a non-tectonic rupture plane that may appear similar to a fault. In this case, the displacement increases with depth due to continued differential subsidence and new deposition. Because 1) very little seismic activity has been recorded in historic time, and 2) instrumental records indicate the level of seismic activity to be very low in St. George Basin, it is difficult at this early stage of investigation to distinguish aseismic growth faults from seismogenic tectonic faults. Positive proof of the presence or absence of seismic activity is needed to distinguish between young tectonic and late stage diagenetic processes in the St. George Basin.

Distant sources are an important cause of earthquakes in the St. George Basin. The modern tectonic plate convergence at the Aleutian trench and the associated Benioff zone is now the dominant distant source of large earthquakes affecting the St. George Basin (see Figure 1-22). The subduction configuration (outlined by Benioff zone) projects beneath the Aleutian Islands from the Aleutian trench. Most of the deepest seismicity occurs at or very near the volcanic line, which suggests that the Benioff zone ceases north of the Aleutian Islands (see Figure 1-22). The focal depths of earthquakes on the Benioff zone north of the Aleutian Islands are likely to be greater than 70 to 95 mi. Earthquake magnitudes at these depths are commonly less than 7-3/4.

The volcanic centers south of the St. George basin within the Aleutian Islands are the most likely sources of volcanic earthquakes in relatively close proximity to the basin (see Figure 1.1). Geologic evidence indicates that two volcanic eruptions occurred on St. Paul Island within the past 10,000 years. In all, 36 eruptive events have been documented during the past 320,00 years (Hopkins, 1975). This suggests that volcanic activity and associated earthquakes are possible in the future, although the recurrence intervals of such events appears to be long. The maximum earthquake is not expected to be greater than about magnitude six to seven based on the world wide record of earthquakes associated with alkali volcanic eruptions.

1.4.1.2 North Aleutian Basin

The North Aleutian Basin geology portrays a relatively stable tectonic environment on the back side of the Aleutian arc. Since late Mesozoic time, there has been only mild folding at the southern edge of the basin near the Alaska Peninsula. The basin consists of an asymmetric, back-arc half graben located to the north of the Alaska Peninsula volcanic island arc (Grantz et al., 1975). In cross secton, the basin is deepest along the northern edge of the Alaska Peninsula and becomes shallower northwestward across the axis of the basin. Subsidence probably ensured during late Mesozoic time during formation of the underlying eroded bedrock platform (Scholl and Hopkins, 1969). The subsidence was accompanied by contemporaneous Cenozoic sedimentation and minor normal faulting but not major diastrophism. The basin is structurally bounded to the north by the Goodnews arch and to the south by the Aleutian volcanic line.

The western offshore limit of the basin is bounded by a change in the structural trends near Unimak Island from west-southwest, typical of the basin, to northwest, based on magnetic, gravity, and seismic profile data. This configuration is consistent with the bend in the offshore regional structural trends in the Bering Sea controlled by the late Mesozoic plate boundary (Marlow et al., 1976a,b).

The North Aleutian Basin structure, in general, lies outside the direct influence of the present plate tectonic collision at the Aleutian trench. The basin lies on the back side of the volcano line forming the backbone of the Alaska Peninsula. This line seems to mark the northern limit of major seismic activity for most of the Aleutian plate convergence. The low seismicity north of this line may signify a lack of deep underthrusting beneath the Alaska Peninsula and the lack of effective penetration of the Benioff zone beneath the north Aleutian basin (AEIDC and ISEGR, 1975). The higher seismicity environment lies south of the volcanic line and underlies the Aleutian shelf and Aleutian trench.

No major faults or structural trends are recognized within the limits of the north Aleutian basin. Most of the evidence for regional compressive forces is preserved in Cretaceous and older rocks that are somewhat faulted and folded (Hoare, 1961). Some of those Mesozoic faults are recognized onshore at the surface, but the influence of tectonic stresses and the recency of activity are far less than those found south of the Aleutian arc (Hoare, 1961). The only major fault trend projecting into close proximity of the north Aleutian basin is the Togiak-Tikchik segment of the Denali fault. A schematic cross section is shown by Figure 1-24.

The southern limit of the basin is bounded by the structural arching of the Alaska Peninsula and its associated active volcanic centers, which may serve as earthquake sources due to volcanic processes.

The greatest distant earthquake source is associated with deformation at and near the modern Aleutian trench tectonic plate boundary located about 160 mi south of the basin. This zone defines the interaction between the North American and Pacific tectonic plates (see Figure 1-24) and dips northward beneath the Alaska Peninsula and the north Aleutian basin at depths probably greater than 70 mi. The seismicity generally deepens from the trench northward. The continuation of this Benioff zone far north of the Alaska Peninsula is not supported by deep seismic events (125 to 250 mi), indicating relatively shallow penetration of the Pacific plate (AEIDC and ISEGR, 1975b). Major earthquakes can be expected from that plate convergence in the future. Some researchers believe that the Alaska Peninsula segment of the plate boundary is overdue for a major earthquake. Based on historic activity, it is considered to be a likely candidate for the next major earthquake in Alaska (Sykes, 1971) with a magnitude in the range of 7.8 to 8.7 (M_b).

The Togiak-Tikchik segment of the Denali fault and the Holitna fault extend northeasterly from Togiak Bay at the far north side of north Aleutian basin. Pratt et al. (1972) suggested a curving offshore extension of the same structural trend through Hagemeister Strait to the 164th meridian in the Bering Sea. However, no extension was found farther west by Marlow et al. (1976a), who point out that the fault may terminate far north of the St. George Basin. The Togiak-Tikchik segment is not considered a probable earthquake source; however, because of its association with the Denali fault, it may have a long recurrence interval between earthquakes of moderate size.

The Holitna fault segment is located northeast of the Togiak-Tikchik fault segment along the Denali fault zone. The fault is a steeply dipping reverse fault that has been traced for about 70 miles and is considered an earthquake-producing structure. No earthquakes of magnitudes equal to or greater than five have been associated with this fault.

The Aleutian arc and Alaska Peninsula consist primarily of accumulations of volcanically derived rocks and large andesitic volcanic centers. At least 60 of those volcanic centers have erupted during the past 10,000 years (AEIDC and ISEGR, 1974b). Usually, the andesitic volcanoes are characterized by violent explosive eruptions. They may be capable of causing moderately large earthquakes. At least ten active volcanoes line the southern edge of North Aleutian Basin (Figure 1-10), any one of which is capable of future volcanic or earthquake activity.

1.4.1.3 Navarin Basin

Because of remoteness and lack of any known significant earthquake activity, there is presently no substantial basis for assessing the seismicity other than its comparative remoteness from the Aleutian underthrust zone. Faults have been interpreted from marine geophysical profiles, but their seismogenic potential is unknown. Comments similar to those regarding a possible "growth fault" origin, as applied to the St. George Basin faults, would appear to apply here also. No major on-land faults are believed to extend into the area.

The shaking produced by the distance earthquake sources in the area of the Aleutian Islands should also be considered in assessing the earthquake risk for any production facilities.

1.4.2 Activity

The seismic activity in the southern Bering Sea is primarily a result of the collisoin of the North American and Pacific plates in the Benioff zone which parallels the trends of the Aleutian Islands. Seismicity drops off in the northern Bering Sea and is caused primarily by local geological faulting.

1.4.2.1 St. George Basin

The seismicity of the St. George basin from 1902 to 1975 is summarized in Figure 1-21. In general the seismic activity directly within the St. George Basin is very low. There seems to be some disagreement in assessing the magnitude of potential earthquakes; OASES assigned a magnitude of 5.9 to the largest event, while others say there is a possibility of having an earthquake with a magnitude up to 7.2 in the St. George Basin. More distant sources are also certainly a consideration with earthquakes up to magnitude 7-3/4 being possible at depths greater than 70 to 95 miles in the Benioff zone north of the Aleutian Islands.

1.4.2.2 North Aleutian Basin

The northern limit of major seismic activity in the North Aleutian basin structures lies along the back side of the volcano line forming the backbone of the Alaska Peninsula. Considering the past seismic history of the area, the Alaskan Peninsula segment of the plate boundary is a potential candidate for a magnitude 7.8 to 8.7 earthquake (Sykes, 1971). The Holitna fault segment northeast of the Togiak-Tikchik fault segment along the Denali fault zone seems to be associated with much smaller earthquakes with none greater than magnitude 5 being expected. Seismic activity in the Bristol Bay region from 1902 to 1975 is shown in Figure 1-21.

1.4.2.3 Navarin Basin

Due to the lack of significant seismic activity in the Navarin Basin, no seismicity maps are included for the Navarin Basin.

1.4.3 Ground Motions

The ground motions expected in the St. George, North Aleutian, and Navarin basins have been summarized in a series of acceleration response spectra in Figure 1-7 to 1-9. Zones for seismicity factors along with API design earthquake are shown in Figure 1-3.

1.4.3.1 St. George Basin

The ranges of 100-year return period peak ground acceleration values from the OASES study are shown in Figure 1-13. The contours indicate that the seismic exposure decreases toward the northwest. In order to develop parameters for seismic design, peak ground acceleration values of 0.1g, 0.125g and 0.14g are estimated as the lower bound, design basis, and upper bound, respectively. Using these acceleration values and spectral shapes developed in the OASES study, design acceleration response spectra were developed. The spectra (for damping = 5%), as shown in Figure 1-7, are for two site conditions: (1) for deep strong alluvium, and (2) for loose or soft material overlying deep strong alluvium. For comparison, the API RP 2A seismic design criteria for this region (Zone 3) is also shown.

1.4.3.2 North Aleutian Basin

The ranges of 100-year return period peak ground acceleration values from the OASES study are shown in Figure 1-13. The trends of the contours show that seismic exposure decreases in the northwesterly direction. In order to develop the parameters for seismic design, peak ground acceleration values of 0.18, 0.24, and 0.3g are estimated as the lower bound, design base, and upper bound, respectively. Using these acceleration values and spectral shapes developed in the OASES study, design acceleration response spectra were developed. The spectra (for damping = 5% shown in Figure 1-9) are for two site conditions: (1) for deep strong alluvium; and (2) for loose or soft material overlying deep strong alluvium. For comparison, the API RP 2A seismic design criteria for this region (i.e., Zone 3) is also shown.

1.4.3.3 Navarin Basin

Based on the low seismicity of Navarin Basin, the 100-year return period peak ground acceleration for the entire basin was estimated to be less than .05g. Using this acceleration value and spectral shapes developed in the OASES study, design acceleration response spectra were developed. The spectra (for damping = 5%) show in Figure 1-8 are for two site conditions: (1) for deep strong alluvium; and (2) for loose or soft material overlying deep strong alluvium. For comparison, the API RP 2A seismic design criteria for this region (i.e., Zone 1) is also shown.

1.5 Ice

Ice can cover the areas of interest in the Bering Sea for approximately eight months out of the year; however, generally ice-free conditions are found from mid-June to mid-October. Ice is therefore an important consideration in the design of structures for the area.

1.5.1 Coverage and Movements

A considerable amount of information exists on the movement of ice within the Bering Sea from satellite data and other observations. One of the more complete studies was presented in 1975 by Potocsky, who analyzed data obtained over 16 years (1954-1970) by U. S. Navy reconnaissance flights, ice breaker reports, and satellite imagery.

Ice edge statistics from the study are plotted in Figure 1-25 for monthly average and extreme values. The ice edge represents ice concentrations greater than 0.1 areal coverage. Presentation of only ice edge information may be somewhat misleading, since the ice edge is so dynamic, moving tens of miles in a single day, seasonal ice coverage varies dramatically. In the St. George Basin and North Aleutian Shelf there may be several consecutive years when no ice is present. For instance, St. George has not had any ice coverage since March 1976. When ice is present in a basin in a given year the ice edge may move back and forth, invading and retreating from an area many times throughout the winter and spring.

Many factors influence the movement and speed of ice floes, including regional winds, waves, currents, ice geometry, and water depth. The results of various studies which have been published show that the ice in the Bering Sea is very dynamic in all but the most highly protected bays and inlets. The ice is moved south throughout most of the winter by strong NE winds. Typical long term motions are on the order of 10 miles per day with maximum velocities of greater than 20 miles per day. Hence, winter operations in most of the Bering Sea will require platforms that can withstand large amounts of ice movements (AOGA, 1980).

1.5.2 Features

Reflecting its vast expanse, a wide variety of ice features may be encountered in the Bering Sea. Significant multi-year ice has not been detected. It is possible that multi-year ice can occasionally drift through the Bering Strait into the Bering Sea (AOGA, 1980).

In general, significant first-year ice features to be expected in the Bering Sea can be divided into the following basic types: sheet ice, broken ice, rubble piles, and pressure riding. Each exists in various forms at various times and in various combinations in the Bering Sea.

Sheet ice is defined as ice of a single thickness. Maximum sheet ice thicknesses for St. Lawrence Island and Port Clarence can be expected to be in the range of 3-1/2 feet. Continual floe fracturing, lead formation and subsequent nila growth and floe movement, however, can result in a wide range of ice thicknesses at any latitude (AOGA, 1980).

Broken ice is ice which is not bonded together in a solid sheet. This type of ice generally exists at an ice edge. The distance between the broken ice edge to solid ice cover is generally no more than about 60 miles during freeze-up. During breakup, however, this distance varies a large amount, depending on wind action.

A pressure ridge is generally defined as a line or wall of broken ice. Ridges are predominantly caused by wind-induced stresses in the ice field. Ice can pile up on top of the sheet ice. Ice pieces also can be accumulated below the waterline, with little or no evidence above. A wide range of ratios of ridge sail height to keel depth thus can occur (AOGA, 1980).

There are indications that the intensity of ridging may vary systematically along the coast. Limited information exists on seasonal and annual changes in ridging. Data on ridging in the Bering Sea are limited.

Recent ridging information comes from laser profiles of ridge sails. The keel depth cannot be accurately estimated from surface data. Means of estimating how frequently rare, large pressure ridges form have not been refined. Data on the bulk properties of first-year compressional and shear ridges and of multi-year ridges are not available, and little is known about the forces involved in the ridging process.

The related information on ice thickness is more important than ice type. No simple method exists to measure ice thickness from the upper ice surface. The best data are from the upward-looking sonar records made during nuclear submarine cruises across the Arctic Basin. These data are at best limited. Some expected ice thicknesses are given in Tables 1-1 to 1-3.

When the polar pack drifts into the shallower waters of the Alaskan continental shelf, the deeper pressure ridge keels can come into contact with the bottom. The result is a series of gouges in the sea floor. In terms of frequency, up to 400 gouges can be found per nautical mile; occasionally, these are deep. Gouges in excess of 10 feet are found off the Alaskan Beaufort coast. Reasonably good documentation of both gouge depths and spatial frequencies are available for the Beaufort coast out to water depths of 115 feet. Some data exist for the Chukchi and Bering Seas but they are not extensive.

Information is not available to accurately estimate the frequency and depth of rare deep gouges or the forces involved in gouging different types of sediments (National Petroleum Council Committee, 1981).

1.5.3 Strength

Ice strength can be divided into two categories: Unit ice sheet strength properties and broken ice mass. It is generally agreed that broken sea ice rubble, usually in the form of first-year pressure ridges, will be a significant factor in the design of offshore facilities and routing of transportation vessels. While considerable effort has been expended to determine uniform ice sheet strengths, investigation of the mechanics and properties of broken ice masses is still in its infancy (AOGA, 1980).

Many investigators from industry, government, and universities have measured unit ice strength properties (e.g., crushing, flexural, shear, adfreeze). For relatively warm, first-year sea ice, such as that in the Bering Sea, crushing and bending strength data are abundantly available (Chevron Oil Field Research Co., 1973; Croasdale, 1971; Kry, 1975; McQuain, 1954; Lee & Simpson, 1954, Shapiro, 1977; Vaudrey, 1977;

and Weeks and Assur, 1967). On the other hand, shear and adfreeze strengths have received little attention, perhaps due to the difficulty of establishing adequate testing procedures. The vast majority of previous ice strength investigations has been either laboratory or small-scale tests. Measurement of large-scale stresses _in situ_ is important, because sea ice is a large crystalline material. Ice behavior analytical models, particularly those including plastic or viscoelastic effects, are applicable to the Bering Sea since it is a region of high deformation and relatively warm ice temperatures (AOGA, 1980).

Behavior of broken ice masses has been considered theoretically using soil mechanics relationships, but properties such as effective cohesion and internal friction are difficult to determine quantitatively. Pressure ridge or ice pileup formation has been modeled mathematically, but only a few field efforts have attempted to determine the degree of consolidation of ridges or ice rubble piles (AOGA, 1980).

Based on probable ice conditions expected in the Bering Sea, ice crushing strengths estimated for the St. George, North Aleutian and Navarin Basins are given in Tables 1-1 to 1-3.

1.5.4 Ice-Structure Interaction

Determination of ice loads on structures is a function of driving force, ice features characteristics, and structural configuration. The driving force is limited by the amount of stress that can be exerted by the wind and current and sustained by the sheet ice. Physical parameters of sea ice, such as sheet thickness, salinity, age, and temperature, are required to determine the strength of the ice. In addition, the general ice feature characteristic, such as sail and keel dimensions, void ratio, and consolidation, are important. The resultant force will depend on ice failure mode, which is primarily a function of the general ice features characteristics and structural configuration (AOGA, 1980).

When the sea surface temperature is within 10°F of the freezing point, the simultaneous occurrence of below-freezing air temperatures and high winds generates waves and spray, which can cause ice to form on the superstructures of vessels and platforms. Existing information on this phenomenon is largely based on observations made by fishing trawlers. Large masses of ice can accumulate rapidly. In extreme cases, the stability of small craft is endangered, but large tankers will have no problem. This hazard will be pronounced in the Bering Sea during times when ice is not present to limit wave conditions. Spray icing will have to be considered in the operation of offshore mobile rigs (National Petroleum Council Committee, November, 1981).

In Tables 1-1 to 1-3, ice loading parameters are presented, including superstructure icing, ice movement velocities, ice thickness/strength, as well as ice loading. Superstructure icing accumulations are to be used to calculate weight added to floating systems.

1.5.4.1 Ice Slamming/Local Pressures

Wave-ice interaction may be the controlling design premises for local loading on Bering Sea platforms. It is assumed the maximum wave height that can be generated with an ice cover and that can transport thick ice pieces is about 15'. Using an ice floe size of 60' diameter and 10' thick, an orbital velocity of 20 fps is calculated from simple energy relationships. Orbital velocity values are extremely conservative because actual transport velocities will be significantly less. It was the consensus of the Environmental Steering Committee to use the range of ice slam velocity values presented in Tables 1-1 to 1-3.

Assuming ice slam velocities, impulse-momentum can be used to calculate local forces.

$$F = \frac{m\,\overline{v}}{t} = \frac{\pi(30)^2\ 10\ (57.6)}{32.2}\ \frac{(6)}{0.2} \qquad (1)$$

$$= 1.5(10^6)\ \text{lbs}$$

Assuming the load is confined to concentrated areas of the structure (5 foot square), the local confining ice pressure used to design stiffeners or reinforcing collars is approximately 450 psi corresponding to a slam velocity of 12 fps. It is suggested that this ice pressure be applied everywhere on the structure from +15' above MWL to -25' below MWL. Local ice pressures associated with ice slam velocities are given in Tables 1-1 to 1-3.

1.5.4.2 Ice Loading/Global Design

If a conservative failure mode (such as continuous crushing) is chosen, the ice loading per unit width of structure can be determined from Korshavin's equation

$$F = If_c\, \sigma_c t \qquad (2)$$

based on unconfined ice compressive strength, consolidated ice thickness, ice type, and ice movement rate for a given ice temperature and salinity. Ice drift velocities in the Bering Sea are sufficiently high to generate maximum ice strength and to reduce the contact factor to its lowest value, $f_C = 0.25$. Assuming an average ice temperature of -3°C and a salinity of 9 ppt, granular and unoriented columnar sea ice strengths can be calculated from API Bulletin 2N Figures 3.2 and 3.3 by multiplying the maximum crystal size values by 30% to account for warm temperatures and high salinities.

Indentation factors of I = 1.2 for granular ice and I = 3.0 for columnar ice can be used to account for confinement. Using rafted ice thicknesses, ice loading values are calculated from Equation (2) and presented in Tables 1-1 to 1-3, assuming each rafted ice floe is composed of 50% granular (top) and 50% columnar (bottom).

The previous calculations are based on rafted ice floes assuming complete consolidation. Another possible scenario is a warm compression ridge moving past a structure leg, failing in double shear. The loading is determined by using the double area based on the assumed ridge cross-section shown in Figure 1-26.

The two ice loading values presented as Parameters 9 and 10 in Tables 1-1 to 1-3 show that rafted ice crushing governs in all but one case; however, it is recommended that for the Navarin Basin rafted ice crushing loads be applied across the entire structure, assuming ice arching effects from a floating rubble field. It is believed that in St. George Basin and North

Aleutian Shelf sufficient clearing will take place, so ice loadings may be applied only across each leg diameter individually.

1.5.5 Wind-Wave-Current-Ice Interaction

While waves generally are not considered a problem on ice covered waters, wave energy can propagate through pack ice to form waves in leads or other areas of open water. The pack ice acts as a selective filter which allows long period wave energy through and dampens out short period energy. As the pack extends both in horizontal coverage and in ice thickness, long period energy also is dampened. Long period swell can be detectable several hundred kilometers into the Antarctic pack ice.

Wave height attentuation through energy loss results from hysteresis in the deflecting ice sheet, dissipation in the boundary layer beneath the ice sheet and hydraulic jumps formed by over-topping water due to wave motion. Thin ice sheets of lengths near 5 times the incident wavelength decay the wave amplitude exponentially, (Ofuya and Reynolds, 1967). The effect is the same as if the waves had propagated into liquid 25 times the viscosity of water. Pack ice of areal extent 20 to 30 times the incident wavelength acts as if it were a liquid with the viscosity equal to 50 times that of water (AOGA, 1980).

Theoretical descriptions of the propagation of waves through pack ice have been attempted since the early 1950's. Actual measurements of wave height or energy after travel through pack ice are few and the work of Robin is probably the most complete. Laboratory simulations have been attempted but modeling both the ice strength and scaling of waves and ice has hampered such work. The slamming of ice on structures by wave action has been studied as related to lighthouses and icebreakers (AOGA, 1980).

1.5.5.1 Storm Wave-Ice Interaction

The following summarizes an analytical study of wave-ice interaction design basis conditions for the Southern Bering Sea.

The assessment of this design condition as summarized in Section 1.5.4.1 was based on a series of experience based subjective judgments made by the study team and Steering Committee.

This initial analytical study was intended to investigate the influence of plausible ranges in the controlling parameters on the design basis impact velocities. An analytical model developed by Mobil Research and Development Corporation (MRDC) was used to analyze the wave-ice interaction phenomenon and determine the ranges in potential impact velocities.

1.5.5.2 Parameter Ranges

Table 1-5 summarizes the combinations of parameters and quantities studied. Table 1-6 defines each of the parameters listed in Table 1-5. Included are parameters that pertain to:

- Ice geometry (width, length, thickness, shape)
- Water conditions (depth, mixing depths)
- Wind conditions (speed)
- Currents (ambient, wind)
- Waves (height, period)
- Steady current coupling (coupling coefficient)

Twenty-seven cases were analyzed, representing plausible ranges in input parameters and representing automated generation of wave and current conditions by the computer code. The program does not allow explicit investigation of the force coefficients and force formulations. These are built-in to the program as described in the next section.

1.5.5.3 Analytical Model

The model can be visualized as illustrated in Figure 1-27. Wind, wave, and current conditions are utilized to describe the forces developed on a floating ice mass of given geometry. Given the forces, velocity contributions of each component (wind, wave, current) are determined. The maximum of each component is summed to determine the resultant velocity of the ice.

Only horizontal velocities are determined. Only a single degree of freedom of motion (horizontal) is considered.

The force (F) formulations important to this problem (a relatively small piece of first-year ice transported by wind, wave, and current) are based on the following:

$$F = K\ A\ V^2$$

where K is a force constant, A is a projected area variable, and V is the wind, wave, or current velocity.

The force constants embodied in the computer program are based on field and laboratory experimental data. For example, iceberg tow studies in which the towing forces and velocities were determined for given sizes and geometrics of ice masses were utilized to determine force coefficients appropriate for steady current velocities. The coefficients were chosen to represent conservative, yet realistic values.

As noted, the computer program allows the user to specify a wind speed and then proceeds to automated computation of wind currents and waves. A Sevdrup, Munk, Bretschneider (SMB) wave forecasting procedure is used to compute the significant wave heights and periods. A classical two-layer, water-wind interaction model is used to determine wind driven currents. A 500 nautical mile fetch is built-into the program to determine automated wave conditions.

The coupling coefficient cited is an operator controlled parameter which is applied to all steady currents. This coefficient provides a means of studying the effects of less than perfect coupling between the steady currents and the ice mass. If the coupling coefficient is assumed to be unity, the ice mass and steady water velocities are assumed to be the same.

1.5.5.4 Results

Table 1-7 summarizes the computed maximum horizontal ice velocities for each of the twenty-seven cases defined in Table 1-5. For the ranges of parameters studied, the velocities range from 4.8 fps to 32.2 fps, compared with a design basis base case of 13.8 fps. Note cases numbers 5 and 18 result in the highest computed velocities, 28.1 and 32.2 fps, respectively. Both of these cases are for the condition where the computer program determines the associated storm heights and currents defined from the input storm wind velocities. In the next section, a discussion is developed on the interpretations of these results.

Case number 1 (Table 1-7) is that intended to replicate the ice wave interaction design basis conditions case defined for this study in Section 1.5.4. The primary elements of input for this case were:

- Ice geometry (20 meter width and length, 3 meter thickness)
- Water conditions (150 meter depth, 10 meter mixing depth; note that in all cases the mixing depth for computing surface currents was chosen to be greater than the ice draft, thus, the ice floe is subject to the full surface current velocities)
- Wind conditions (choosen as 30 minute average at 10 meters elevation; note computer code does not decrease wind velocity to velocity at free surface)
- Currents (ambient of 0.5 meters per second and wind current of 1.0 meter per second)
- Waves (5 meter height and 10 second period)
- Coupling coefficient (assumed to be 1.0)

The computed maximum horizontal velocity is 13.8 fps; compared with an initial design basis condition assessment of 12 fps, very good agreement.

The computed maximum horizontal velocity of 13.8 fps is composed of the following velocity contributions:

- Wind drag, 3.0 fps
- Wave velocity, 4.4 fps
- Wave drift, 1.5 fps
- Wind current, 3.3 fps
- Ambient current, 1.6 fps

The motions contributed by the waves account for the largest velocity component (5.9 fps, or 43 percent of the total). The motions contributed by the currents account for the second largest velocity component (4.9 fps, or 36 percent of the total). The motions contributed by the wind drag account for the remainder (3.0 fps, or 22 percent of the total).

Case number 25 (Table 1-7) is that intended to simulate possible wave-ice interaction conditions during design basis oceanographic conditions in the Southern Bering Sea. The primary elements of input for this case were:

- Ice geometry (20 meter width and length, 3 meter thickness)
- Water conditions (150 meter depth, 10 meter mixing depth)
- Wind conditions (130 mph)
- Current (4 fps)
- Waves (80 feet height, 14 second period)
- Coupling coefficients (1.0)

The computed maximum horizontal velocity is 25.7 fps; compared with an initial design basis condition assessment of 12 fps, a large discrepancy.

The computed maxium horizontal velocity of 25.7 fps is composed of the following velocity contributions:

- Wind drag, 3.3 fps
- Wave velocity, 17 fps
- Wave drift, 1.4 fps
- Wind current, 3.3 fps
- Ambient current, 0.7 fps

Again, the wave accounts for the largest velocity component, 17 fps, or 66 percent of the total. For a wave height of 70 feet and 13 second period, all other variables held constant, the computed maximum horizontal velocity is 24.5 fps with the wave contributing 15.9 fps or 65 percent of the total.

Figure 1-28 presents a "bow-tie" graphical portrayal of the sensitivity of the changes in the base case resultant ice velocity with changes in key parameters. The presentation is given as the percentage change in the resultant velocity (ordinate) with the percentage change in a given parameter. Within the ranges portrayed by these results and focusing on the positive changes in the resultant velocities, the parameters that have the largest influence on the results are in order of importance: currents, wind speed, wave height, and ice size. Note that changes in these parameters of + or -100 percent have only a 10 to 40 percent change in the resultant parameters. The resultant velocities appear to be relatively insensitive to plausible changes in the parameters within these ranges.

1.5.5.5 Discussion

The results developed in these analyses indicate that the initially assessed design basis ice-wave interaction impact velocity of 12 fps, and an associated range of 10 to 20 fps, generally fall within the range determined from the foregoing analyses. The principal exception regards the definition of the storm wind, waves, and currents that should accompany or be combined with a winter ice-edge breakup condition.

The analyses indicate storm wave heights in the range of 14 to 25 meters and wind currents in the range of 0.5 to 1.7 meters per second associated with storm wind velocities in the range of 50 to 100 knots. The great majority of the values used in the assessment were for wave heights in the range of 5 meters. Consideration of the environmental conditions that could lead to storms resulting in breakup of the ice edge (severe winter storms) implicate wave heights, wind currents, and wind velocities in excess of the majority of the values utilized in the original assessment. This is an important topic for further consideration in development of realistic design criteria for the ice-wave interaction in the Southern Bering Sea.

Another topic warranting further consideration is that associated with the analytic computer model utilized for these analysis. The model represents an important first-generation step foward in analyzing the interaction of waves and ice. The model represents a combination of rudementary physics, hydrodynamic theory, field and laboratory empirical results, and judgments. The model has been intended to develop results that are "realistic, yet conservative". The model is being applied to conditions that generally lie outside the primary ranges for which it was developed (iceberg-wave interaction in moderately severe oceanographic conditions). No calibrations have been done on the model.

Undoubtedly, more complete and realistic analytic models can be developed. The question is whether such models are warranted given the present understanding of the parameters, given quantities required as input to such a model, and given the basic understanding of the phemenona controlling wave-ice interaction. Physical model testing, combined with improved analytical modelling is a promising next step in the developments.

Another element warranting consideration regards the structure being impacted by the ice. Thus far, a rigid structure that must fully absorb the impact has been assumed. This assumption can either be conservative or unconservative depending on the characteristics of the structure and of the ice. Compliance of the structure can either amplify or reduce the impact. The crushing of the ice as it impacts the structure can lead to substantial reductions in the energy absorption requirements of the structure itself. Three-dimensional motions of the ice (rotation, vertical) add further complexities to this consideration. Again, a combination of physical model testing and analytical modeling is suggested as a promising avenue for development of more realistic analyses.

1.6 Design Basis Conditions

The design basis conditions and the associated ranges in these conditions are intended to provide a realistic basis for:

- Screening alternative concepts of production systems and components.
- Determining the projected or estimated costs associated with the most viable of these systems.
- Identifying major engineering problems, constraints, and information needed to allow development of production systems to go forward without undue delays or costs.

The design basis conditions should not be confused with design criteria. Design criteria consist of two primary components: (1) Environmental Loading Criteria, and (2) Structural and Foundations Strength Criteria. The intended combined effect of these two components is a production system that represents an equitable balance of costs and risks for a particular development, type of system, and specific location.

This study has not attempted to explicitly develop either an integrated system of environmental loadings and structural-foundation criteria nor a risk-cost balanced criteria. Rather, it has utilized generally accepted design methods and criteria from other similar locations, such as the North Sea, adapted for the unique environmental conditions in the Southern Bering Sea.

The study consulting team and the participants Steering Committee feel that the design basis conditions and the associated ranges in these conditions (identified as lower basis and upper basis) bracket plausible design criteria for projected operations and production systems in the Southern Bering Sea. Due to the unusual multiple environmental loading threats which are present in this area (storms, earthquakes, ice and wave-ice) and the large uncertainties presently associated with characterization of these loading environments, it is suggested that interpretitve weighting be given to the upper basis design conditions and associated system costs.

1.6.1 Method Used to Develop

The information summarized in this report represents the synthesis of information currently available in the literature. The two major sources of environmental information were oil and gas industry-sponsored research and development reports and documents available from Federal Agencies. The background information was summarized by the consultants and then reviewed by the industry Steering Committee to provide the current set of recommended design basis conditions.

1.6.2 Soils

The distribution of soil types is known in only a very broad sense from seismic reflection profiles and core samples as is shown in Figures 1-4 and 1-12. The soil properties of the St. George Basin are much better known than either the Navarin or North Aleutian Basins, especially in deep water. On the inner continental shelf of the Bering, surface sediments are primarily granular in nature decreasing in size away from the coast and with depth. On the outer shelf sands still predominate, while in deep water clayey silts become more prevalent.

The information from this report is intended as a guide to help in assessing the probable strength and stiffness of soils in the areas of interest in the Bering Sea. The engineering properties of the soils quoted here were based on bottom sediment information and core samples. Two basic soil types have been identified, cohesive and granular. The cohesive soils shown Figure 1-5 reflect the crust-like behavior of the soil; three different profiles for soft, medium, and stiff clay have been plotted to provide a bound on the expected soil conditions. In Figure 1-6, reflecting the variations in density and grain size characteristics in the soil profile, three different curves have been plotted for granular soils to bound the range of soils expected; observe that for loose to medium silts and sands and for medium to dense sands, liquefaction may be a problem. Additional information on the expected soil properties is also summarized in Tables 1-1 to 1-3. It is assumed that no significant changes in properties are caused by environmental conditions. It should be noted that permafrost is not expected to be a problem in the areas of interest.

An important related consideration for the soil properties is the problem of potential geologic hazards. Liquefaction under seismic and storm wave conditions may be a problem in both the St. George and North Aleutian Basins. Based on bathymetric information given in Figures 1-4 and 1-10, and the unstable sediments present in the St. George Basin, slope stability may present a problem in that area and to a lesser extent in the other two basins. Both the St. George and Navarin Basins should be considered areas where gas-charged sediments may prove to be a hazard.

It should be understood that the information in this study is general in nature. Site-specific data must be determined for any proposed installation.

1.6.3 Oceanographic

Tables 1-1 to 1-3 contain a summary of the design basis oceanographic conditions in the St. George, Navarin and North Aleutian Basins. The lower and upper bound water depth range of interest was set at 300 to 600 feet in the St. George and Navarin Basins with a design basis at 450 feet, and a sensitivity range of 200 to 350 feet in the North Aleutian Basin with a design basis of 300 feet.

Weather is very changeable over the Bering Sea. Severe storms in the Bering Sea often cover a large area and are dominated by intense Aleutian low pressure. The months from fall through spring are characterized by traveling cyclic disturbances which traverse the region as shown in Figures 1-13 to 1-15. Winds of over 100 mph can be expected each year in each of the basins. Expected extreme wind speeds are given in Figure 1-17. The design basis wind speeds in Tables 1-1 to 1-3 are 130 mph for both the St. George and Navarin Basins and 120 mph in the North Aleutian Basin.

Waves are a direct result of the storms in the area. Extreme and operational conditions can be expected to be similar to those in the North Sea. From Tables 1-1 to 1-3, the following design basis wave conditions are given: St. George Basin, wave height 85 feet and period 15 seconds; Navarin Basin, wave height 80 feet and period 14 seconds; and North Aleutian Basin, wave height 78 feet and period 15 seconds. Similar wave heights as well as wave steepness are indicated for the API design table in Figure 1-2.

Information about currents in the Bering Sea is especially critical in predicting ice movements and hydrodynamic forces. Such information is relatively scarce for the area. From the sources available in addition to the sensitivity range, the following design basis currents from Tables 1-1 to 1-2 were selected: St. George and Navarin Basins, surface current speed 4 fps and bottom current speed 0 fps; and North Aleutian Basin, surface current speed 5 fps and bottom current speed 2 fps.

Water level rises due to surges and tides are addressed in Table 1-1 to 1-3. Design basis surge and tide heights were set at 7, 5 and 12 feet in the St. George, Navarin and North Aleutian Basins, respectively.

1.6.4 Earthquakes

The OASES study (1980) provided much of the primary basis for establishing the earthquake design criteria. The seismic activity is considerable greater south of the Aleutian chain than in the Bering Sea, with seismicity dropping off as one proceeds northward in the areas of interest. Within the Bering Sea itself, the North Aleutian Basin is expected to experience the largest earthquakes, followed by the St. George Basin; few large earthquakes are expected in the Navarin Basin.

In developing the earthquake acceleration response spectra in Figures 1-7 to 1-9, both near-field events caused by local faulting as well as far-field events from distant sources were considered. Curves were developed for 5 percent of critical damping for two soil conditions and various ground accelerations; it should be noted for each of the curves that a strong alluvium layer is postulated. For contrast, the API RP 2A accelerations from Figure 1-3 are also included based on the appropriate zone classification. From Tables 1-1 to 1-3, the design basis conditions for ground acceleration, ground velocity, and ground displacement were given respectively as: St. George Basin, 0.125 g, 4.25 ips, and 0.5 in.; Navarin Basin, 0.05 g, 1.5 ips, and negligible; North Aleutian Basin, 0.24 g, 8.0 ips, and 1.5 in. Upper and lower ranges of values for the basins are also included in those tables.

It should be cautioned that local faulting should be considered in the local site evaluation when assessing geological hazards.

1.6.5 Ice

Ice can cover the Bering Sea from mid-October to mid-June. Ice conditions can vary considerably from year to year as indicated in Figure 1-25. Ice loads upon structures may take three different forms: icing which adds additional weight and mass to the superstructure, quasi-static pressure loading caused by drifting sheet ice or pressure ridges, and dynamic loads produced by ice slamming. In addition to the thickness of the ice, the ice forces are also influenced by the physical structure of the ice as well as whether the ice has been formed by pressure ridges or rafting. The design basis conditions for the three basins which involve icing, slam velocity, local ice pressure, and crushing strength are summarized in Tables 1-1 to 1-3.

Only those conditions which were dependent on the rafted floe thickness and ridging thickness were expected to vary from basin to basin.

Contrasting with a rafted floe thickness of 3 feet and a ridging thickness of 15 feet in the St. George and North Aleutian basins, more severe conditions were expected in the Navarin Basin with 8 feet and 20 feet for the corresponding values, respectively.

For the design basis wave-ice interaction conditions, it was assumed the maximum wave height that can be generated with an ice cover and that can transport thick ice pieces is 15'. Using an ice floe size of 60' diameter and 10' thick, an orbital velocity of 20 fps is calculated from simple energy relationships. Orbital velocity values are extremely conservative because actual transport velocities will be significantly less. It was the consensus of the Steering Committee to use the range of ice slam velocity values presented in Tables 1-1 to 1-3.

To complement the values proposed for the ice slamming velocities and the local pressures, additional background material was developed in this study. Ice slam velocities in the range of twice the design basis condition of 12 fps were possible, however, dependent primarily upon the wave height, indicating a need for caution in assessing local ice pressures and careful consideration of the wave conditions to be used in combination with the ice edge break-up conditions.

1.7 Major Engineering Information Needed

One of the primary objectives of this study was to identify major engineering information needed pertaining to environmental conditions and criteria to allow development of the Bering Sea Basins to go forward without undue and significant delays, costs, and risks.

This evaluation has been based on the information which has been developed during this study and in the subsequent findings of the studies of production system concepts. The evaluation has also been based on the study documented in the report by the Alaskan Oil and Gas Association, "Recommended Research Projects for the Alaskan Bearing Sea" (January 1980).

The major engineering information needs cited here are those the study team has identified as being critical to the production system concepts identified for feasibility study. Information needs for other system concepts have not been identified.

Some of the major engineering information needs cited here are fulfilled during the normal course of exploration and production operations, e.g., site hazards surveys and geotechnical studies. They have been cited both for the sake of completeness and because of unusual or particularly hazardous environmental conditions in portions of the Southern Bering Sea.

Engineering information needs relative to environmental conditions identified herein fall into two categories: (1) information needed so that appropriate engineering strategies can be developed for production systems to adequately withstand the environmental conditions, and (2) information needed so that the uncertainties associated with the characterization of the environmental conditions can be reduced, thereby allowing the use of less conservative factors-of-safety and allowing significant savings in first and long-term costs.

Table 1.4 identifies 22 specific efforts intended to provide information for design and siting of production systems in the Southern Bering Sea. These efforts have been organized into five categories: (1) soils, (2) oceanography, (3) earthquakes, (4) ice, and (5) design criteria. The general background of the efforts cited will be discussed in the following sections.

1.7.1 Soils

Data on near surface sediments in the Southern Bering Sea has been gathered chiefly by the USGS. While the data is relatively sparse in this very large area, when coupled with geophysics and geologic information, a general appreciation of sediment distributions and characteristics is possible.

Similarly, data on subsurface sediments in the Southern Bering Sea has been gathered chiefly by the industry. However, only a few deep soil borings have been obtained and analyzed.

While this information has been sufficient for purposes of this study, the information is not sufficient for lease or site specific purposes. Because production system concepts and costs are significantly impacted by the soil conditions that must support these systems, careful assessment of local soil and geologic conditions is important. Thus, site specific and route specific soil borings and surveys have been cited in Table 1.4.

A wide variety of geohazards have been identified in the Southern Bering Sea (Marine Board, The National Research Council, "Environmental Exposure and Design Criteria for Offshore Oil and Gas Structures" 1980). Principal geohazards include sediment instabilities, sediment erosion and deposition gas in sediments, shallow faults, and ice scouring. In general, these geohazards are restricted to certain small areas of the lease areas and are not widely distributed.

Specific geohazards at any one location may impact lease tract selection or siting of production systems and components, e.g., pipelines. Pre-lease sale identification of potential geohazards is the responsibility of the Federal Government as a part of their prelease sale Environmental Impact assessment and reporting. Post lease sale identification and evaluation of specific hazards is the responsibility of the operators as a part of the pre-exploratory and pre-development data collection and engineering processes. Collection and evaluation of pertinent site geohazards data is an operator requirement under current Federal regulations. Thus, geohazards and bathymetric surveys and studies for platform sites and pipeline routes were identified as engineering information needed.

1.7.2 Oceanography

The direct force producing effects on storm winds, waves and current acting on production systems in the Southern Bering Sea are one of the three principal environmental loading hazards (ice and earthquakes are the other two). Extreme conditions produce controlling loading effects that have important implications regarding overall strength requirements for these systems, floating or fixed, concrete or steel. Nominal, or more frequently occurring wind, waves and currents, produce important effects on operations, constructions, transportation and resupply. Production system fatigue and dynamic response considerations also are of importance.

In addition to the direct force producing effects of extreme and nominal condition oceanographic conditions are the indirect effects on seafloor soil conditions. Scour and other forms of instabilities (general and local) as well as potentially important changes in the engineering properties of the soils themselves can be developed by both nominal and extreme oceanographic conditions.

Because of the pervading and potentially important influences of the oceanographic environment on production systems in the Southern Bering Sea, and because of a general lack of measured data on this environment, a major engineering information need is to increase the data base on wave, current and wind conditions. Hence, wave, current, and wind measurements have been cited in Table 1-4.

Measurements provide a necessary but not sufficient basis for characterizing design and operating oceanographic conditions. Generally, this is due to the limited times and geographic coverage and instrumentation-operation limitations that are inherent in practical or feasible measurement programs. Thus, analytical models and numerical analyses become a means of understanding and extending measured data. Such efforts are referred to in Table 1-4 in the form of design and operating oceanographic conditions. Production system cost and reliability implications are large for such efforts.

Of particular importance in the efforts regarding oceanography of the Southern Bering Sea is the recognition and assessment of ice cover and ice edge dynamics. Ice cover will act to limit the fetch or areas over which the storm winds can generate waves and currents. Ice edge dynamics will have important implications on the changes in energy and frequency content of storm wave systems in the ice edge area, on the break-up characteristics of the ice itself (influencing the wave-ice interaction forces on structures), and on the movements of the ice edge during storms (influencing the operational restrictions on some production system components, e.g., transport tankers). These are high priority considerations that will have important cost and reliability implications for production systems in the Southern Bering Sea, particularly those in Navarin and St. George Basins.

1.7.3 Earthquakes

The OASES study (1980) has been the major source of information concerning earthquakes used to characterize the seismic exposure of the Southern Bering Sea. Due to a lack of measured data on the seismotectonic characteristics of this area, this study had to rely on extrapolations of measured data and understanding obtained from other analogous areas and synthesized within the analytical framework of a state-of-the-art seismic exposure analysis model. The seismic exposure model utilized was composed of three primary elements: (1) source characterizations (location, type, capability, and event recurrence rate); (2) transmission characteristics (the change of key strong ground motion description parameters with distance and the change of energy and frequency content of the ground motions with distance and azimuth from the sources); and (3) local site effects (influence of local geologic and soil conditions in modifying ground motion characteristics.

All of these factors influence the seismic exposures in the Southern Bering Sea to some degree, and the dominant factors vary from region to region. In the St. George and North Aleutian areas, seismic exposure is dominated by the dipping Benioff zone along the Aleutian Arc. Source zone locations and event characterizations are generally well understood. However, the transmission characteristics are not well understood. In the OASES study, measurements from Japan formed a primary portion of the data base utilized to describe these transmission characteristics. The data and analysis base were both limited.

In the northern portions of the Southern Bering Sea, there are only a few recorded significant earthquakes, indicating that the overall exposure likely is low. However, the lack of recording stations in the area and the general lack of shallow and deep geophysics data has not allowed the identification of local fault structures or seismic sources. Thus, there is a need for a seismic network and offshore geophysics/geologic studies which would permit accurate assessment of source characteristics in the northern portion of the Southern Bering Sea. Again, due to the potentially important force producing effects on production systems proposed for this area, such efforts have potentially important cost and reliability implifications for these systems.

For the foregoing reasons, three general efforts pertaining to earthquakes and evaluation of seismic exposure have been identified in Table 4. These are (1) local geologic and site effects, (2) source surveys and assessments (including transmission assessments), and (3) seismic exposure evaluations (based on the data gathered from the foregoing effects).

1.7.4 Ice

Ice loadings pose a principal environmental hazard in the northern portions of the Southern Bering Sea. Ice loadings can control or dominate both the local and global elements of production systems and components of such systems. Ice coverage can be anticipated in the northern portions of this area for seven to eight months of each year and infrequently in the southern portions for four to six months. Thus, ice poses important operating considerations and restraints.

Some good data is available in the Southern Bering Sea on ice coverage, movements, growth, thickness, features, and strength characteristics. The second two efforts, design and operating conditions, are intended to utilize the information from the first two efforts in data analysis and numerical hindcast/simulation studies in reliably characterizing ice operating and force conditions important to production system design (strength, ductility) and operations connect, construction, resupply, etc.).

Ice floes transported by storm waves and interacting with surface piercing structural elements (wave-ice interaction) is a controlling design premise for the local loadings of Southern Bering Sea Production Systems. The initial design premises for ice slamming local pressures have been based on judgment. The implications of this design premise is that small diameter members cannot be used in the free-surface zone (-25 ft to +15 ft). Well conductors, pipeline risers, mooring system elements, bracing elements for deck sections, and other similar surface piercing elements must either be grouped and protected within very large diameter members or legs or terminated above and below the free surface zone. Other implications include the necessity for moving the equipment (floating-moored structures) out of the areas potentially influenced by ice-wave interaction.

Thus, wave-ice interaction design premises are having a major influence upon the structural configurations and operations that can be considered.

In development of the ice loading design premises for global loading effects, it has been assumed in two of the study areas (North Aleutian Shelf, St. George Basin) that ice loadings will occur on all surface piercing elements, assuming that the ice interacting with these elements will act as ice interacting with a single member. In the third study area (Navarin Basin), the design premise has been taken that ice crushing loads are applied across the entire structure. Thus, the outside or projected area of the entire structure is taken for computation of the ice loadings.

For the structure concepts studied thus far in two of the three basins, the design basis ice loadings are governing the global loadings on the steel plat forms. Ice loadings are resulting in total lateral forces in the range of 50 percent greater than those associated with the design oceanographic conditions (80-foot design wave with 4 feet per second design current).

Thus, ice interaction with multileg structure design premises are having a major influence upon the structural configurations and operations that can be considered.

Ice force design conditions for global and local analyses of surface piercing structures and structural elements involve large uncertainties. Further, these design conditions are influenced by other design, construction, and operations constraints. Surface piercing member size and spacings are influenced by the other sources of loadings, well drilling considerations, production requirements, and construction equipment as examples. Costs are also a consideration in the determination of ice force design conditions.

The wave-ice interaction problem involves the following primary considerations:

- Definition of wind, wave, and current conditions (e.g., the probabilities of experiencing combinations of wind speeds, wave heights, and current speeds that could result in ice-edge break-ups and transport of this ice into the structures).

- Definition of ice conditions (e.g. the probabilities of experiencing combinations of ice thickness, plan area, and strength that could result in ice impact forces on the structure).

- Definition of ice-wave interaction conditions (e.g., given the wind speeds, wave heights, and current speed combinations, characterize how the ice floe combinations interact to define ice impact velocities).

- Definition of ice-structure interaction conditions (e.g., given the ice floe condition and ice impact velocities, and the structure response characteristics, characterize the probabilities of experiencing local pressures and gross forces).

Considerations associated with ice interaction with multileg structures have been outlined in the API Bulletin for Planning, Designing and Constructing Fixed Offshore Structures in Ice Environments. Analytical and model studies are suggested to guide the formulation of design basis conditions.

Analytical studies have been reported in the literature and model study results summarized. Experience with multileg platforms in ice areas has been reported in the literature. These results indicate that ice jamming is possible, depending upon the characteristics of the ice, of the structure, and of the force environments acting upon the ice. The limited experience and data which is available does not indicate that the forces experienced during jamming conditions exceed those experienced during other similarly severe ice loading conditions (individual multi-leg loadings for example).

Future work could include:

- An analytical and model study of ice-interaction with multileg structures would be proposed. The analytical study would investigate the influence of different leg sizes, spacings, orientations, and numbers in conjunction with different ice floe sizes, characteristics, and driving forces. A three-dimensional finite element analysis and computer model are envisioned for use in the analytical study.

- A companion physical model study of ice-interaction with multileg structures would be proposed. The physical model study would investigate the same range of geometrical and ice conditions outlined in the analytical model study. A qualitative understanding is expected from the physical model study to aid in the development of the quantitative results from the analytical study.

Results from the first two components of this study are expected to significantly improve qualitative and quantitative understanding of the expected ranges of ice-structure interaction forces and stresses for the types of drilling and production systems being considered for the Southern Bering Sea. However, large uncertainties are expected to exist due both to natural variabilities (e.g., ice floe size, ice strength, etc.) and interaction modeling uncertainties. These uncertainties must be accommodated with adequate strength in the structures and with appropriate operating procedures and equipment.

1.7.5 Design Criteria

Design criteria are intended to be an integrated system of analytical methods, specified quantities and parameters, allowable stresses and factors of safety, and quality assurance procedures that when combined and applied to the engineering of a production system will result in a system that has desirable qualities of safety, cost, and operability. Table 1.4 identifies three component efforts relating to design criteria: (1) environmental, (2) structural, and (3) foundation.

The production systems study for the Southern Bering Sea has identified some potentially unique considerations relating to design criteria. The principal unique considerations relating to environmental design criteria are those associated with the following:

- Multi-hazard nature of this environment with potentially significant loadings and effects from storms, earthquakes, ice, and storm-ice interaction conditions.

- Intense nature of the threats from environmental loading and effects, both from extreme and nominal conditions.

- Very large uncertainties and variabilities associated with the environmental conditions and forces.

- Unique structural, foundation, and operational requirements and elements involved in the alternative production system concepts.

- Very large investments, high operating costs, and significant resources associated with production systems in this environment.

These unique considerations implicate a careful and deliberate effort to integrate existing technology, experience from similar operating environments, data gathering and analysis results from programs such as discussed here, to result in cost effective and reliable production systems that will permit efficient development of the resources of the Southern Bering Sea.

REFERENCES

SECTION 1.0

ACI Committee 357, Title No. 75-72, "Guide for the Design and Construction of Fixed Offshore Concrete Structures, 1978.

AEIDC and ISEGR (Arctic Environmental Information and Data Center; and Institute of Social, Economic, Governmental Research), "The Bristol Bay Environment, A Background Study of Available Knowledge," Department of the Army, Alaska District, Corps of Engineers, Contract No. DACW 85-73-C-0068, 1975.

Alaska Oil and Gas Association, Lease Sale Planning and Research Committee, "Recommended Research Projects for the Alaskan Bering Sea," January 1980.

American Petroleum Institute, "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," API RP 2A, 12th edition, 1981.

American Petroleum Institute Task Group on Fixed Offshore Platforms in Ice Environments, "API Bulletin for Planning, Designing and Constructing Fixed Offshore Structures in Ice Environments," Second Draft, May 1981.

Carlson, P. R., and Karl, H. A., "Sea Floor Geologic Hazards, Sedimentology, and Bathymetry: Navarin Basin Province, Northwestern Bering Sea," U. S. Geological Survey Open File Report 81-1217, 1981.

Gardner, J. V. and Vallier, T. L., Faulting, Unstable Sediments, and Surface Sediments in Southern Bering Sea, Outer Continental Shelf (abs.): U. S. Geological Survey, in press, 1977.

Gardner, J. V., Vallier, T. L., Dean, W. E., Kvenvolden, K. A., and Redden, G. D., "Sedimentology and Geochemistry of Surface Sediments and the Distribution of Faults and Potentially Unstable Sediments, St. George Basin Region of the Outer Continental Shelf, Southern Bering Sea," U. S. Geological Survey Open File Report 79-1562, 1979.

Grantz, A., Holmes, M. L., Kososki, B. A., "Geologic Framework of the Alaskan Continental Terrace in Chukchi and Beaufort Seas," U. S. Geological Survey, Open File Report 75-124, 43 p., 1975.

Hoare, J. M., "Geology and Tectonic Setting of Lower Kuskowin-Bristol Bay Region, Alaska," American Association of Petroleum Geology Bulletin, Vol. 45, No. 5, p. 594-611, 1961.

Hopkins, D. M., "Fault History of Pribilof Islands and Its Relevance to Bottom Stability in St. George Basin, in Environmental Assessment of the Alaskan Continental Shelf," Principal Investiigators Reports, July-September 1975, Vol. 2, p. 395-400 and year ending March, 1976, Vol. 13: U. S. Department of Commerce, National Oceanic and Atmospheric Administration, 1975.

Marlow, M. S., McLean, H., Vallier, T. L., Scholl, D. W., Gardner, J. V., and Powers, R., "Preliminary Report on the Regional Geology; Oil and Gas Potential and Environmental Hazards of the Bering Sea Shelf South of St. Lawrence Island, Alaska," U. S. Geological Survey Open File Report 76-785, 1976a.

Marlow, M. S., Scholl, D. W., Cooper, A. K., and Buffington, E. C., "Structure and Evolution of the Bering Sea Shelf South of St. Lawrence Island," The American Association of Petroleum Geologists, Vol. 60, No. 2, 1976b.

National Petroleum Council's Committee on Arctic Oil and Gas Resources, "U. S. Arctic Oil and Gas," Draft Report, November 16, 1981.

National Research Council, National Academy of Sciences, Committee on Offshore Energy Technology of the Marine Board, Assembly of Engineering, "Environmental Exposure and Design Criteria for Offshore Oil and Gas Structures," Washington, D. C., May 1980.

Offshore Alaska Seismic Exposure Study, Woodward-Clyde Consultants, prepared for Alaska Subarctic Operators' Committee, March, 1978.

Pratt, R. M., Rutstein, M. S., Walton, E. W., and Buschur, J. A., "Extension of Alaska Structural Trends Beneath Bristol Bay, Bering Shelf, Alaska," Journal of Geophysical Research, Vol. 77, No. 26, pp. 4994-4999, 1972.

Scholl, D. W., Buffington, E. C., and Marlow, M. S., "The Structure and Origin of the Large Submarine Canyons of the Bering Sea, in Marine Geology," Elsevier Publishing Company, Amsterdam, Vol. 8, p. 187-210, 1975a.

Scholl, D. W., Buffington, E. C., and Marlow, M. S., "Plate Tectonics and the Structural Evolution of the Aleutian-Bering Sea Region, in the Geophysics and Geology of the Bering Sea Region," Geological Society of America Special Paper 151, pp. 1-31, 1975b.

Scholl, D. W., and Hopkins, D. M., "Newly Discovered Cenozoic Basins, Bering Sea Shelf, Alaska," The American Association of Petroleum Geologists Bulletin, Vol. 53, No. 10, p. 2067-2078, 1969.

Sharma, G. D., Naidu, A. S., and Hood, D. W., "Bristol Bay: Model Contemporary Graded Shelf," American Association of Petroleum Geology Bulletin, Vol. 56, No. 10, p. 2000-2010, 1972.

Shepard, F. P., Submarine Geology, 2nd edition, New York, Harper, 557 p., 1963.

Sykes, L. R., "Aftershock Zones of Great Earthquakes, Seismicity Gaps, and Earthquake Prediction for Alaska and the Aleutians," Journal of Geopysical Research, Vol. 76, No. 32, p. 8021-8041, 1971.

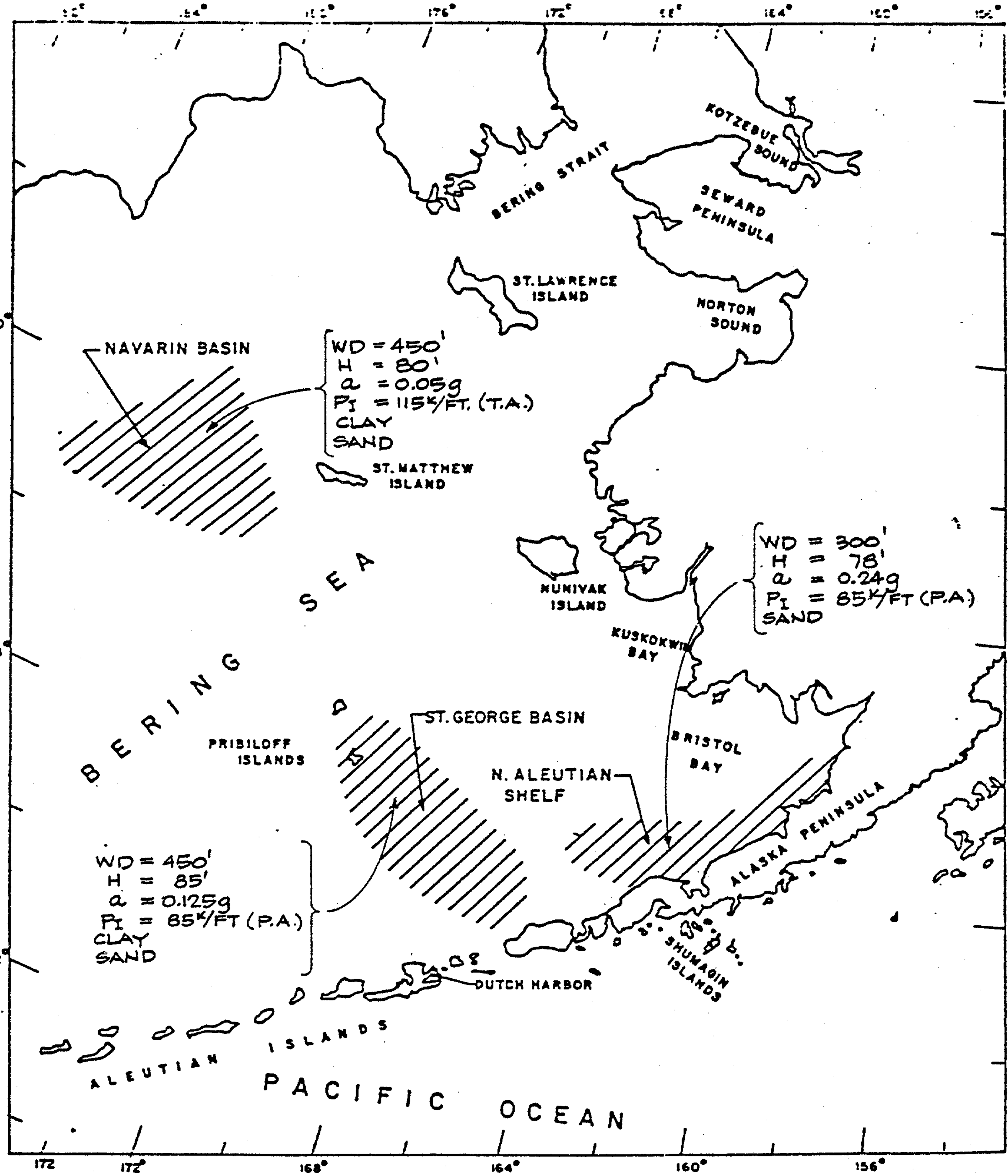
United States Geological Survey, "Appendices to Requirements for Verifying the Structural Integrity of OCS Platforms," OCS Platform Verification Program, October 1979.

United States Geological Survey, "Commentary on Requirements for Verifying the Structural Integrity of OCS Platforms," OCS Platform Verification Program, October 1979.

United States Geological Survey, "Operating Procedures for the OCS Platform Verification Program," October 1979.

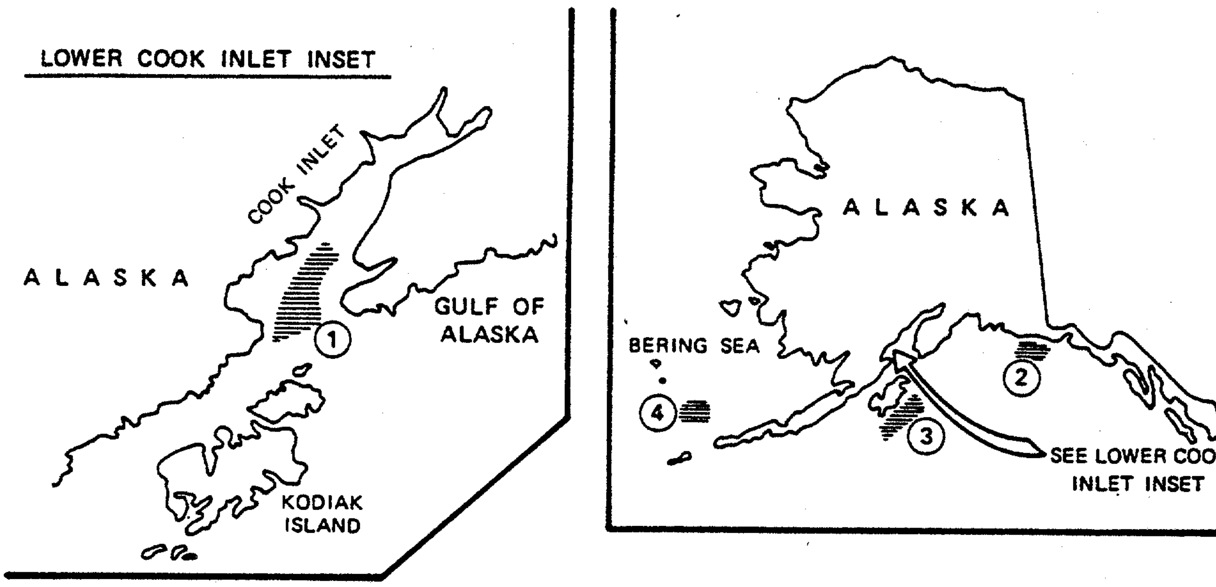
United States Geological Survey, Outer Continental Shelf Order No. 8, "Platforms and Structures," OCS Platform Verification Program, October 1979.

United States Geological Survey, "Requirements for Verifying the Structural Integrity of OCS Platforms," OCS Platform Verification Program, Ocrober 1979.



STUDY BASINS AND ENVIRONMENTAL SUMMARY

FIGURE 1-1



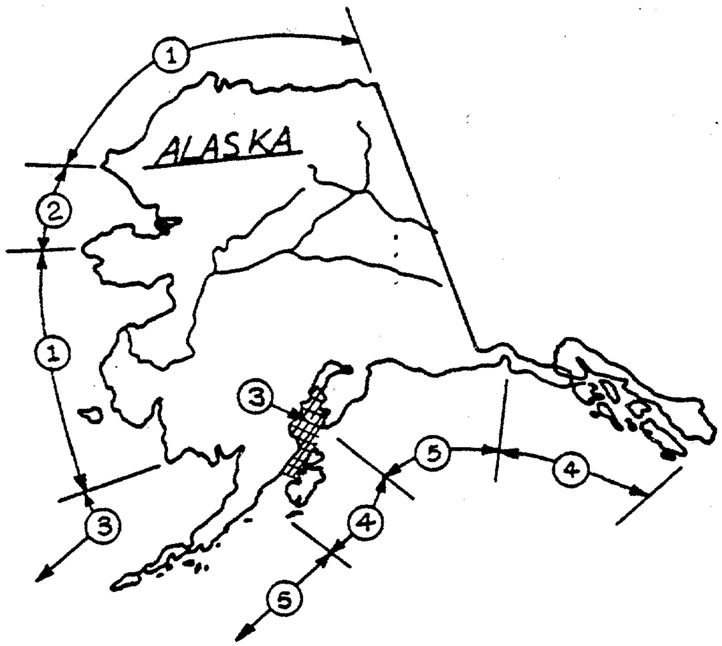
WAVE PARAMETERS
For Ten Areas in United States Waters

	Wave Height				Wave Steepness	Reference Level Deck Clearance*	
	Reference Level		Guideline				
	ft	*m*	*ft*	*m*		*ft*	*m*
Offshore Gulf of Mexico	70	21.3	(see Fig. 2.8-2)		1/12	48	14.6
Offshore Alaska							
1. Lower Cook Inlet	60	18.3	50-70	15-21	1/13	56	17.1
2. Icy-Bay Gulf of Alaska	100	30.5	90-120	27-37	1/15	80	24.4
3. Kodiak Shelf-Gulf of Alaska	90	27.4	80-110	24-34	1/15	72	21.9
4. Bering Sea/Bristol Bay	85	25.9	75-95	23-29	1/13	63	19.2
Offshore California							
1. Santa Barbara Channel	45	13.7	40-50	12-15	1/16	38	11.6
2. Outer Banks	60	18.3	55-70	17-21	1/15	44	13.4
Offshore Atlantic Coast							
1. Georges Bank	85	25.9	75-95	23-29	1/12	59	18.0
2. Baltimore Canyon	90	27.4	80-100	24-30	1/12	62	18.9
3. Georgia Embayment	75	22.9	65-85	20-26	1/12	53	16.2

*Above MLW in Atlantic; above MLLW in Pacific.

API DESIGN WAVE HEIGHTS

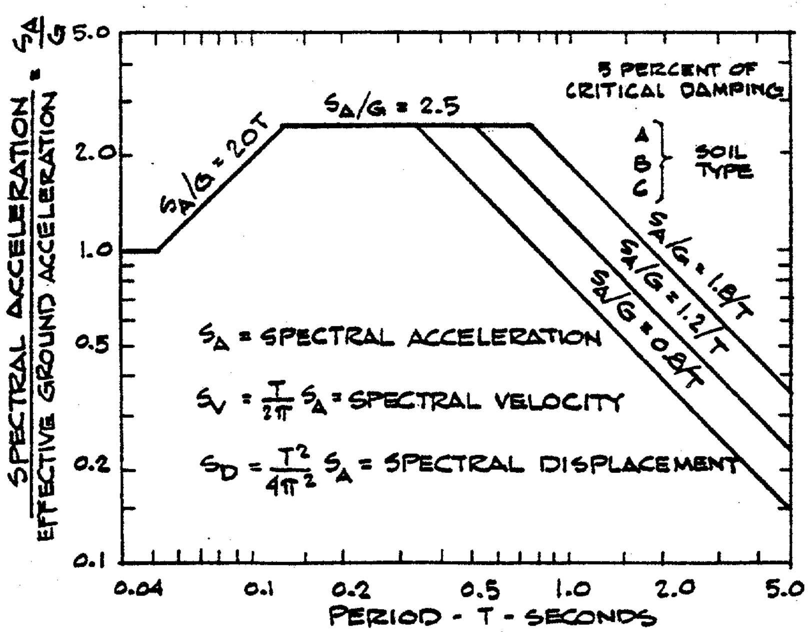
FIGURE 1-2



Z = 0   1   2   3   4   5
G = 0   0.05   0.10   0.20   0.25   0.40

Where Z = Zone or relative seismicity factor given in Fig. 2.3.6-1.

G = Ratio of effective horizontal ground acceleration to gravitational acceleration.



SOIL TYPE

A. ROCK - CRYSTALLINE, CONGLOMERATE, OR SHALE-LIKE MATERIAL GENERALLY HAVING SHEAR WAVE VELOCITIES IN EXCESS OF 3000 FT/SEC (914 M/SEC).

B. SHALLOW STRONG ALLUVIUM - COMPETENT SANDS, SILTS AND STIFF CLAYS WITH SHEAR STRENGTHS IN EXCESS OF ABOUT 1500 PSF (72 kPa), LIMITED TO DEPTHS OF LESS THAN ABOUT 200 FEET (61 M), AND OVERLYING ROCK-LIKE MATERIALS.

C. DEEP STRONG ALLUVIUM - COMPETENT SANDS, SILTS AND STIFF CLAYS WITH THICKNESSES IN EXCESS OF ABOUT 200 FEET (61 M) AND OVERLYING ROCK-LIKE MATERIALS.

API DESIGN EARTHQUAKES

FIGURE 1-3

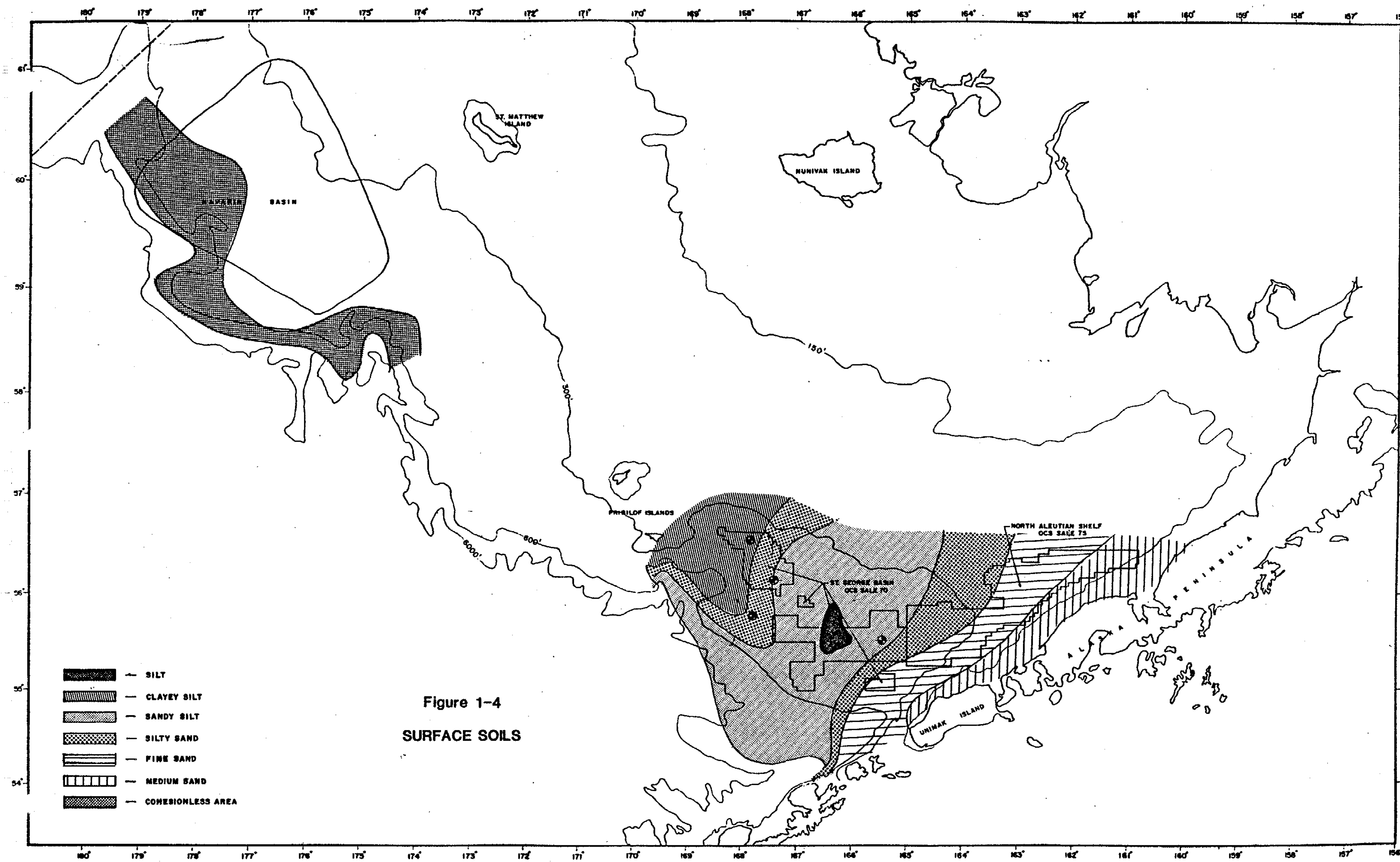


Figure 1-4

SURFACE SOILS

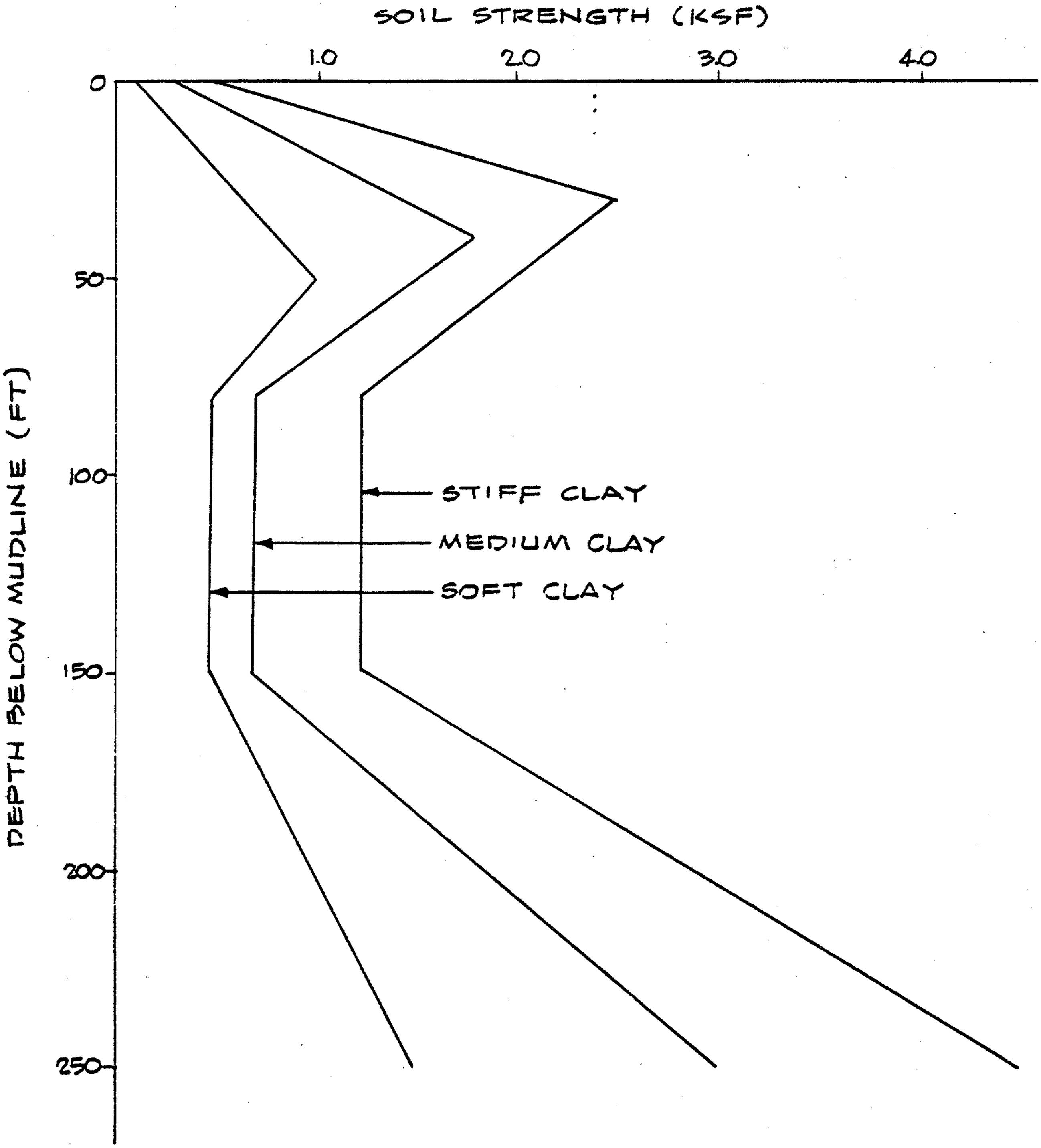


FIGURE 1-5 SOIL TYPE A, COHESIVE - ASSUMED SOIL STRENGTH FOR ST. GEORGE AND NAVARIN BASINS

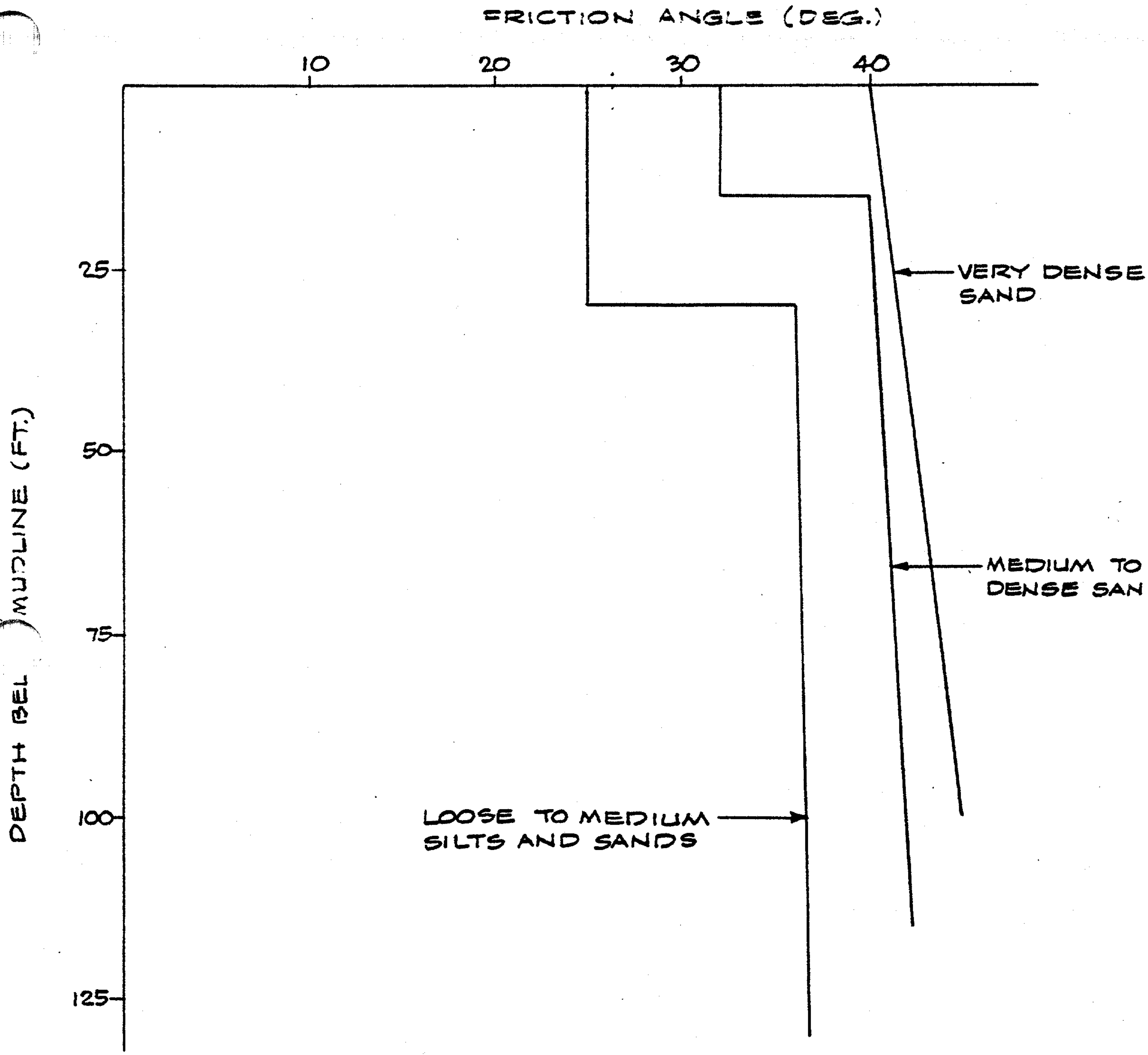
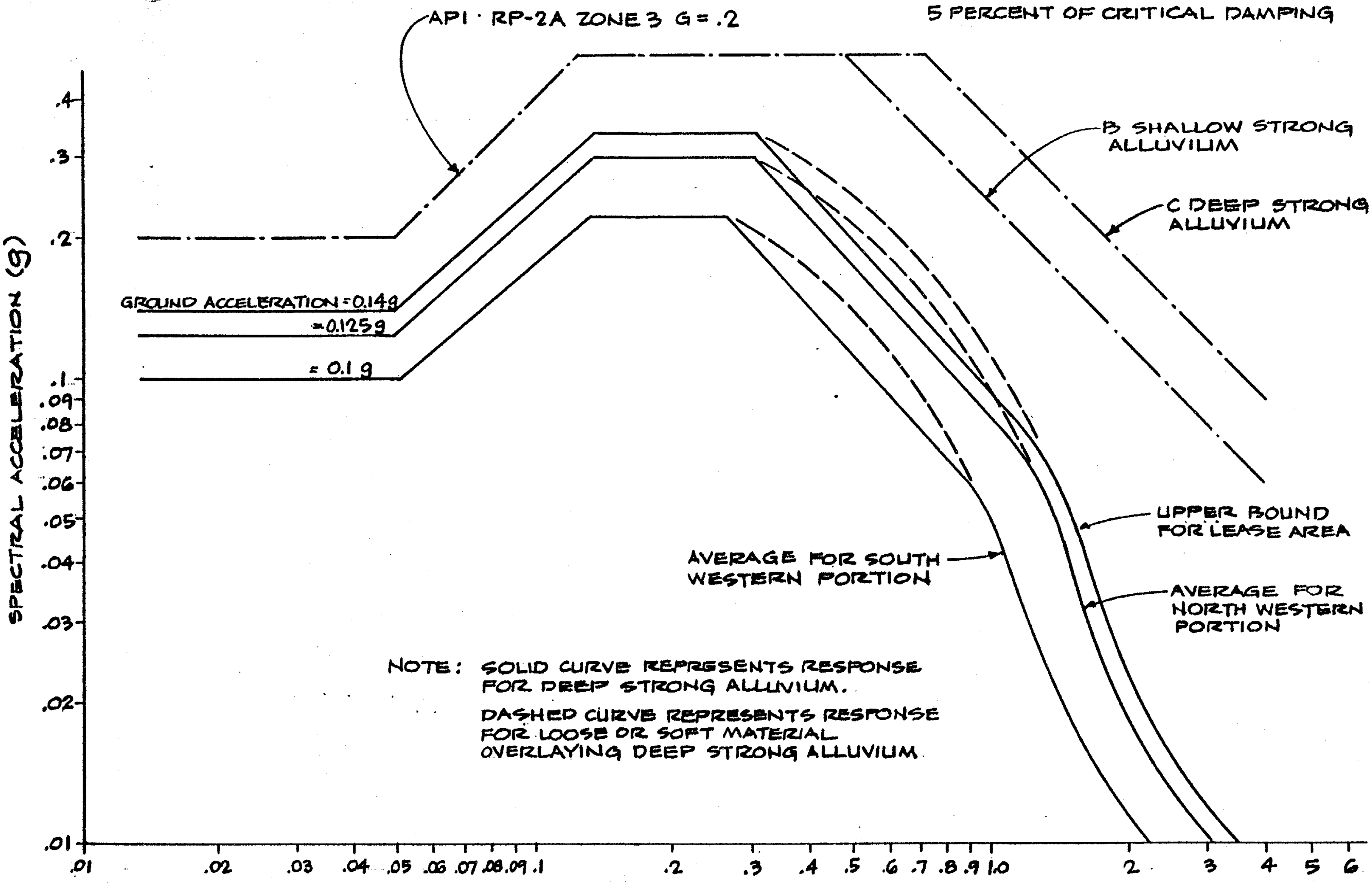
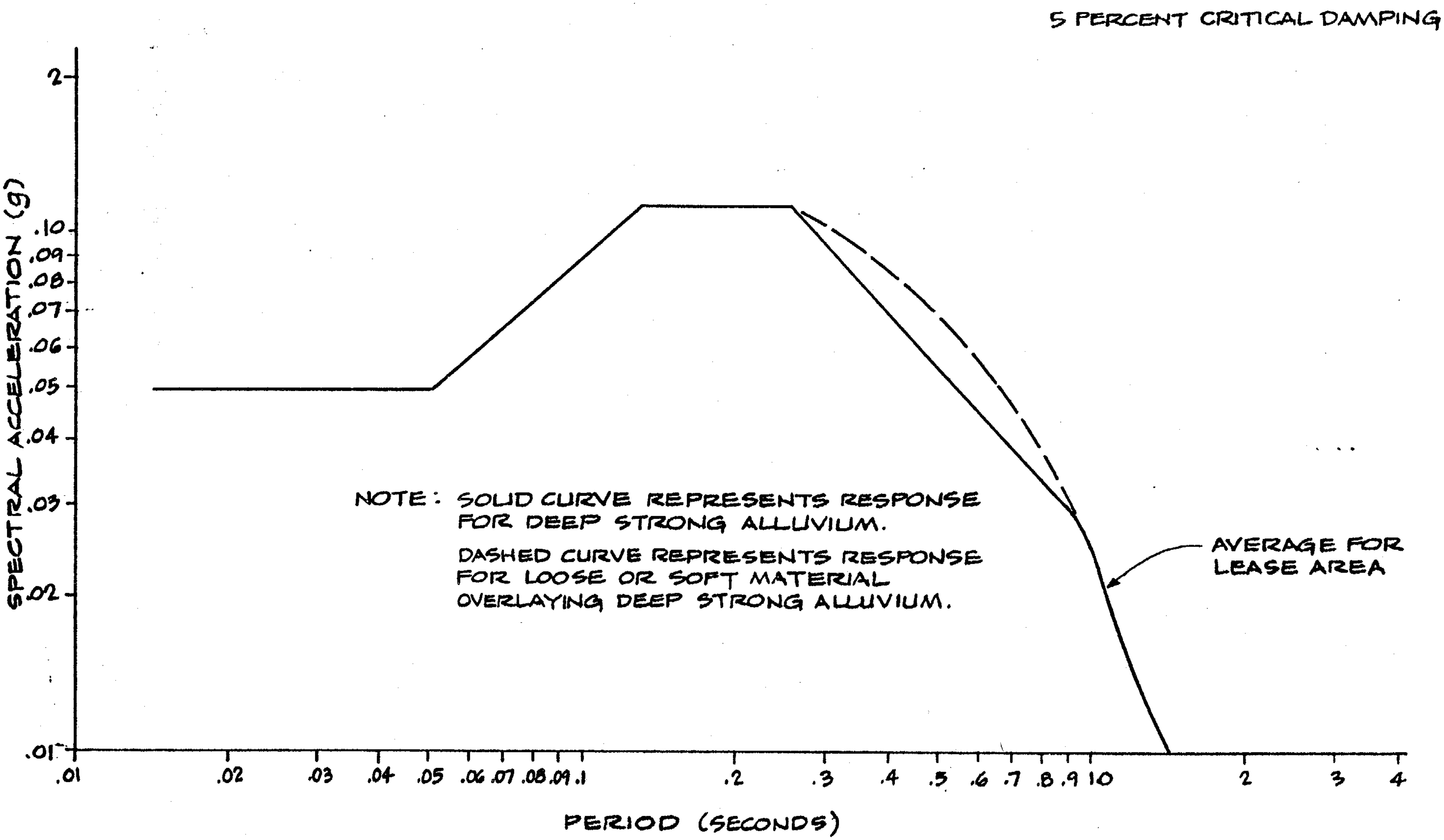


FIGURE 1-6 SOIL TYPE B, GRANULAR - ASSUMED SOIL PROPERTIES FOR ST. GEORGE BASIN, NAVARIN BASIN AND THE NORTH ALEUTIAN SHELF



ACCELERATION RESPONSE SPECTRUM - ST. GEORGE BASIN

FIGURE 1-7



– ACCELERATION RESPONSE SPECTRUM – NAVARIN BASIN

FIGURE I - 8

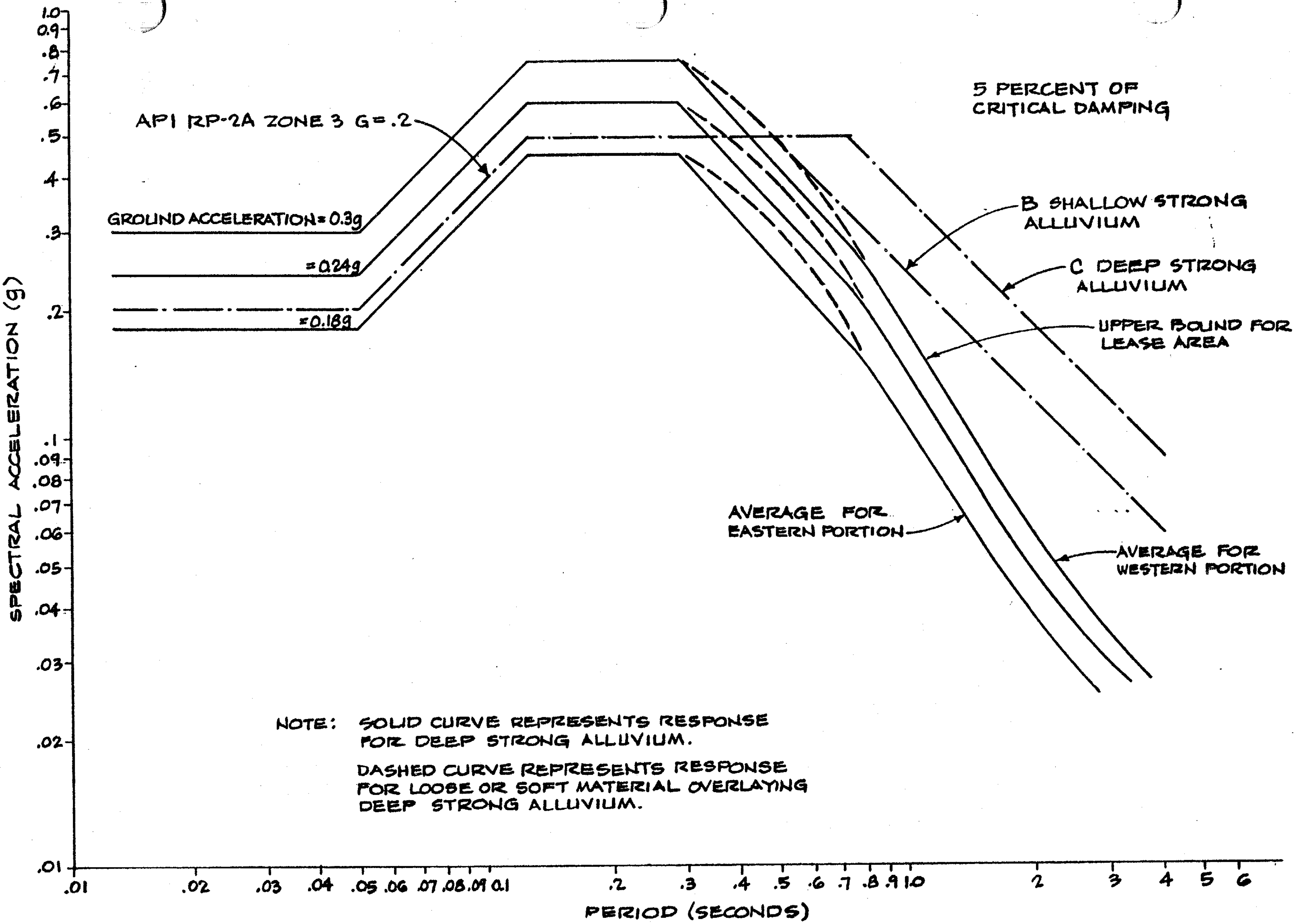


FIGURE 1-9 ACCELERATION RESPONSE SPECTRUM - NORTH ALEUTIAN SHELF

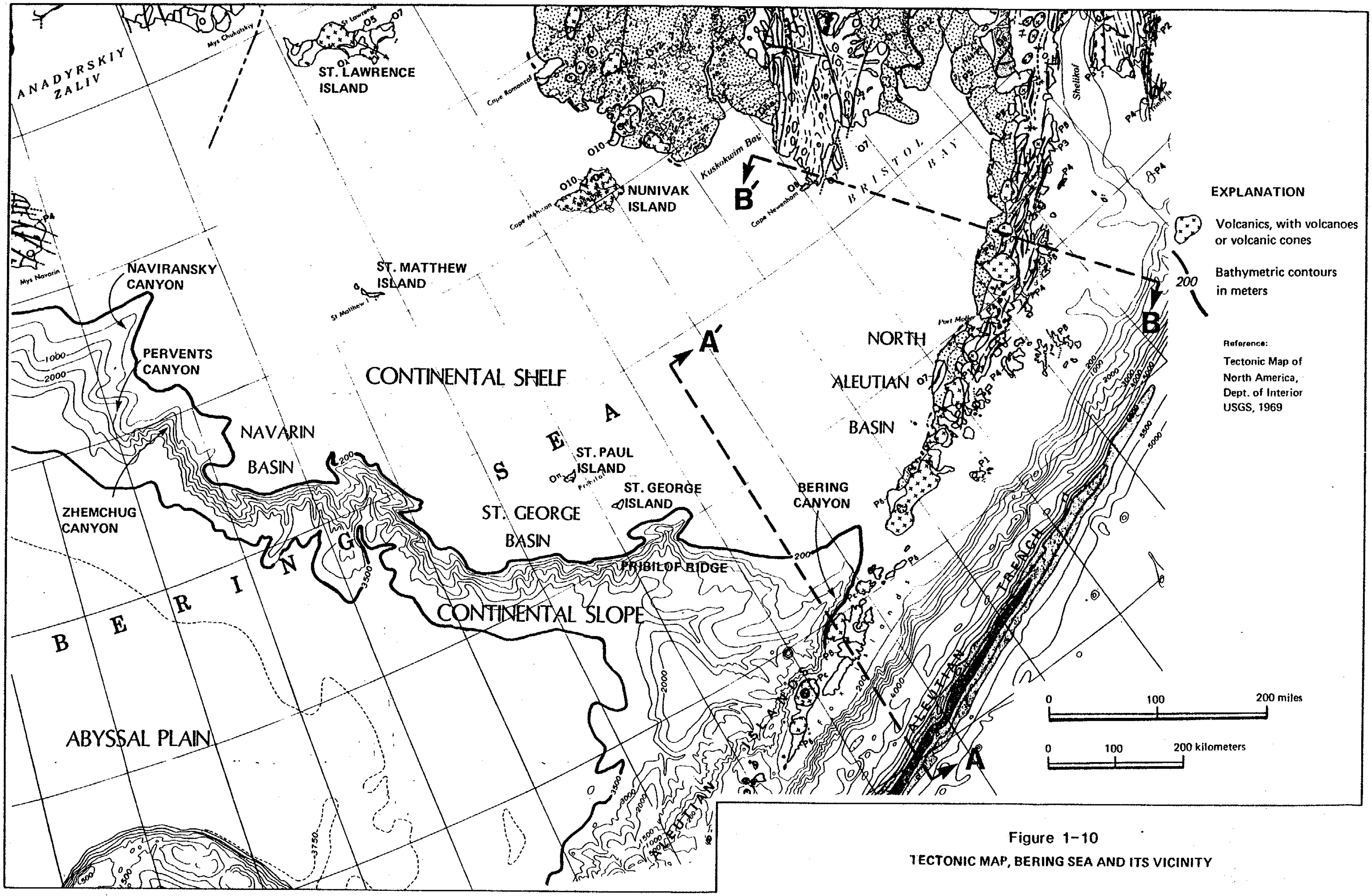
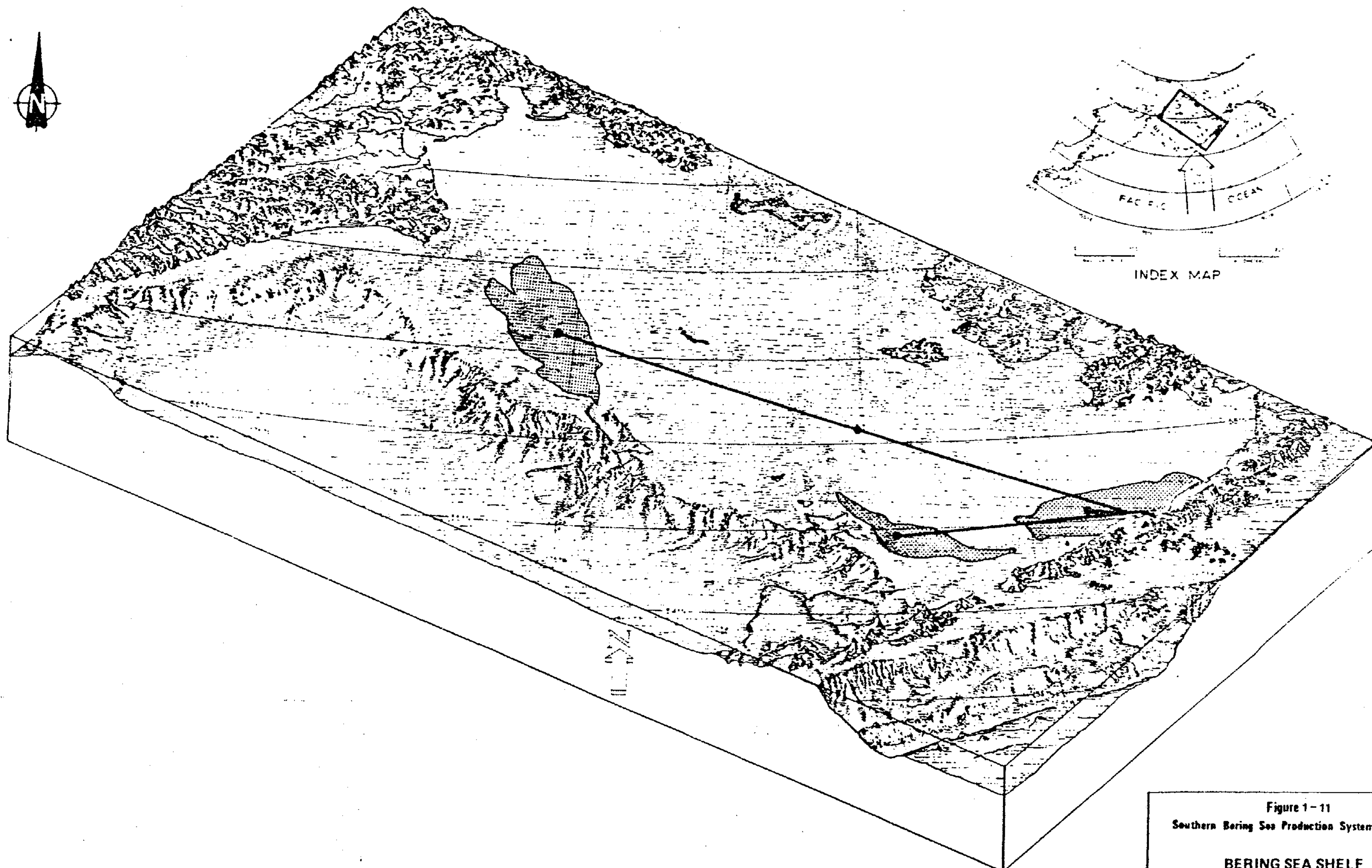


Figure 1-10

TECTONIC MAP, BERING SEA AND ITS VICINITY

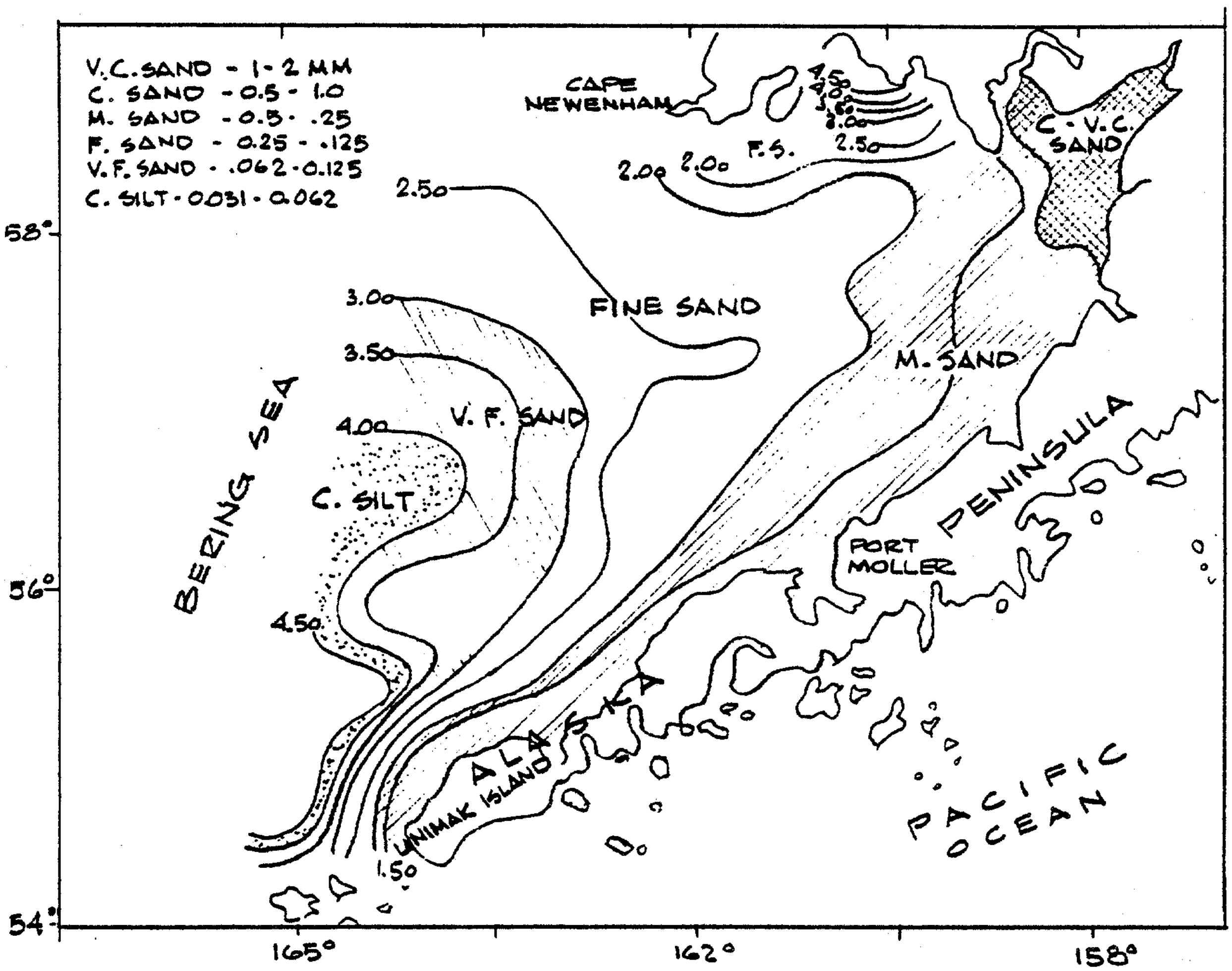


ORTHOGRAPHIC DRAWING—
U. S. GEOLOGICAL SURVEY
OPEN FILE MAP 77–591

Figure 1 – 11
Southern Bering Sea Production Systems Study

BERING SEA SHELF
AND SURROUNDINGS

Job 15154 | Feb. 1982



DISTRIBUTION OF SURFACE SEDIMENT SIZE OVER THE SOUTHEASTERN BERING SHELF

SOURCE: GEOLOGICAL OCEANOGRAPHY OF THE BERING SHELF BY G. D. SHARMA

FIGURE 1-12

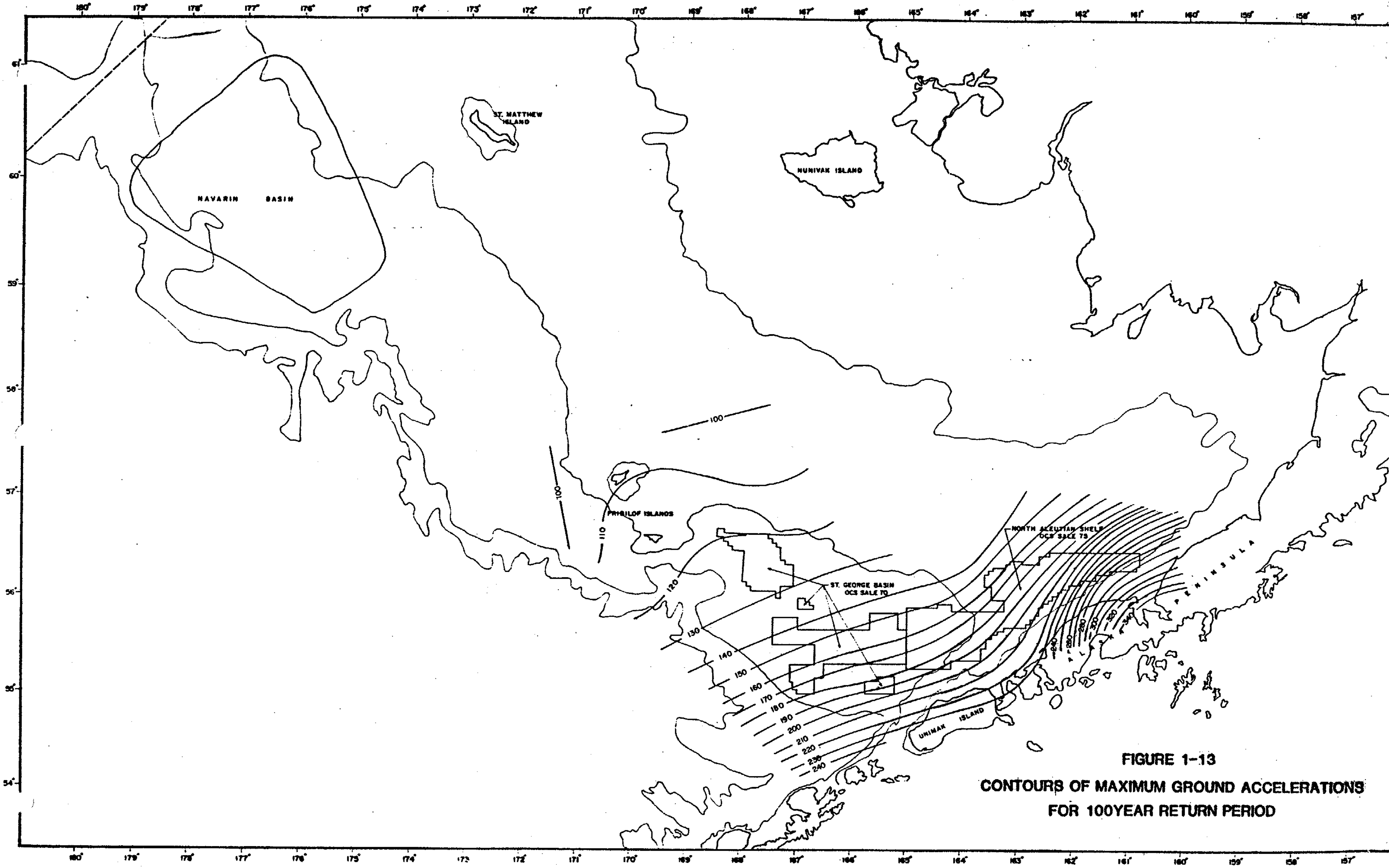
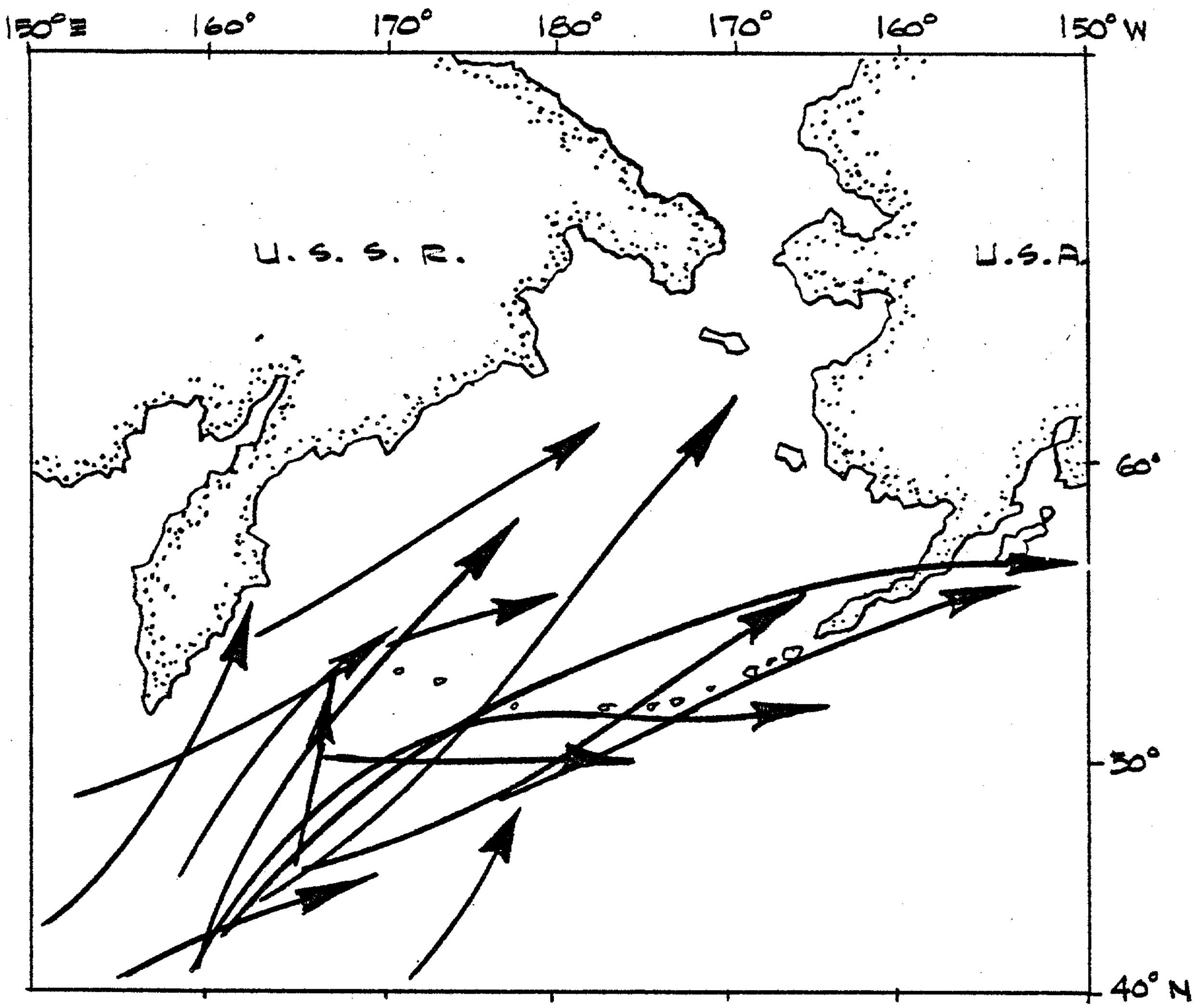


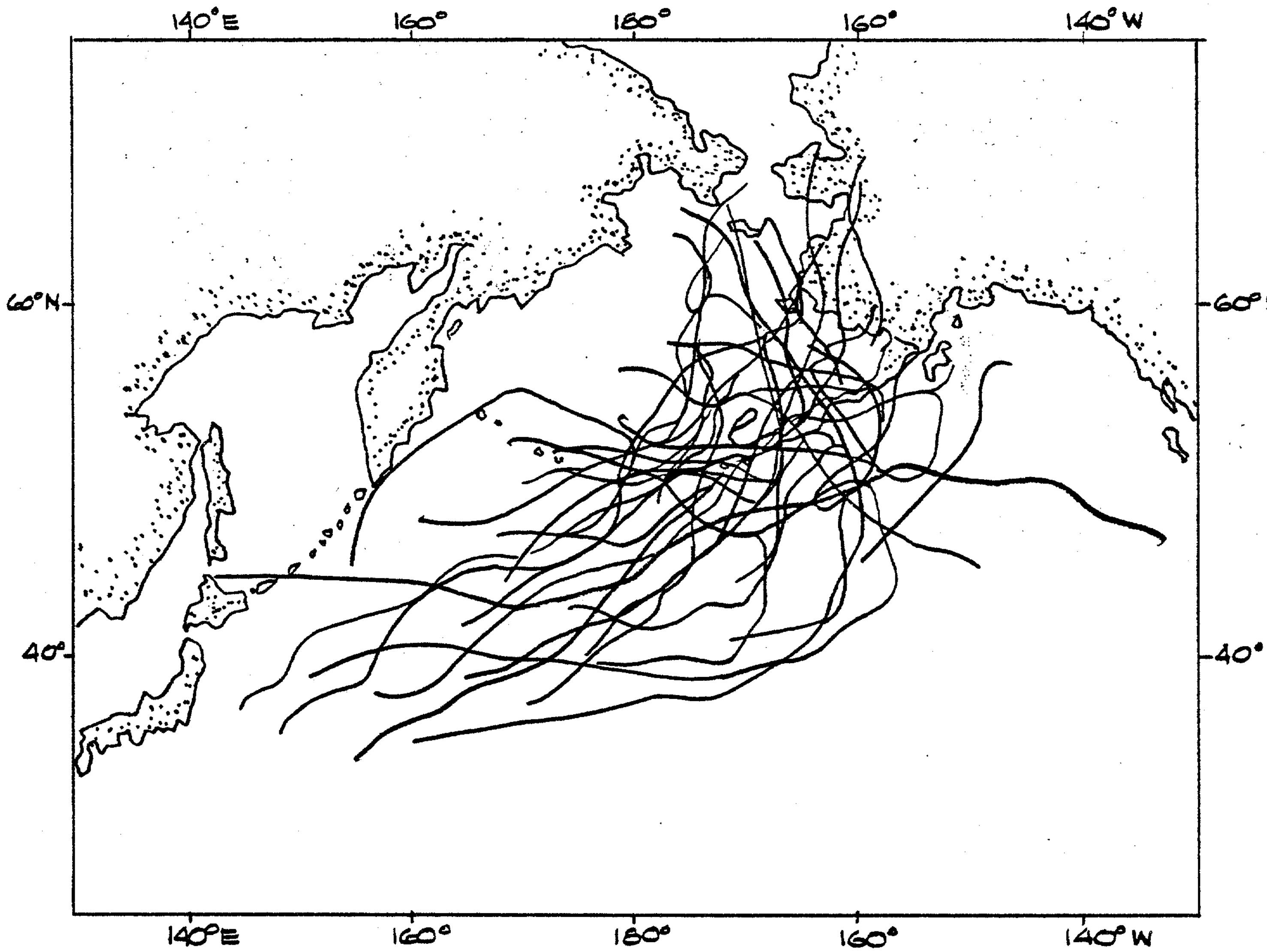
FIGURE 1-13
CONTOURS OF MAXIMUM GROUND ACCELERATIONS
FOR 100YEAR RETURN PERIOD



BERING SEA PRIMARY STORM TRACKS

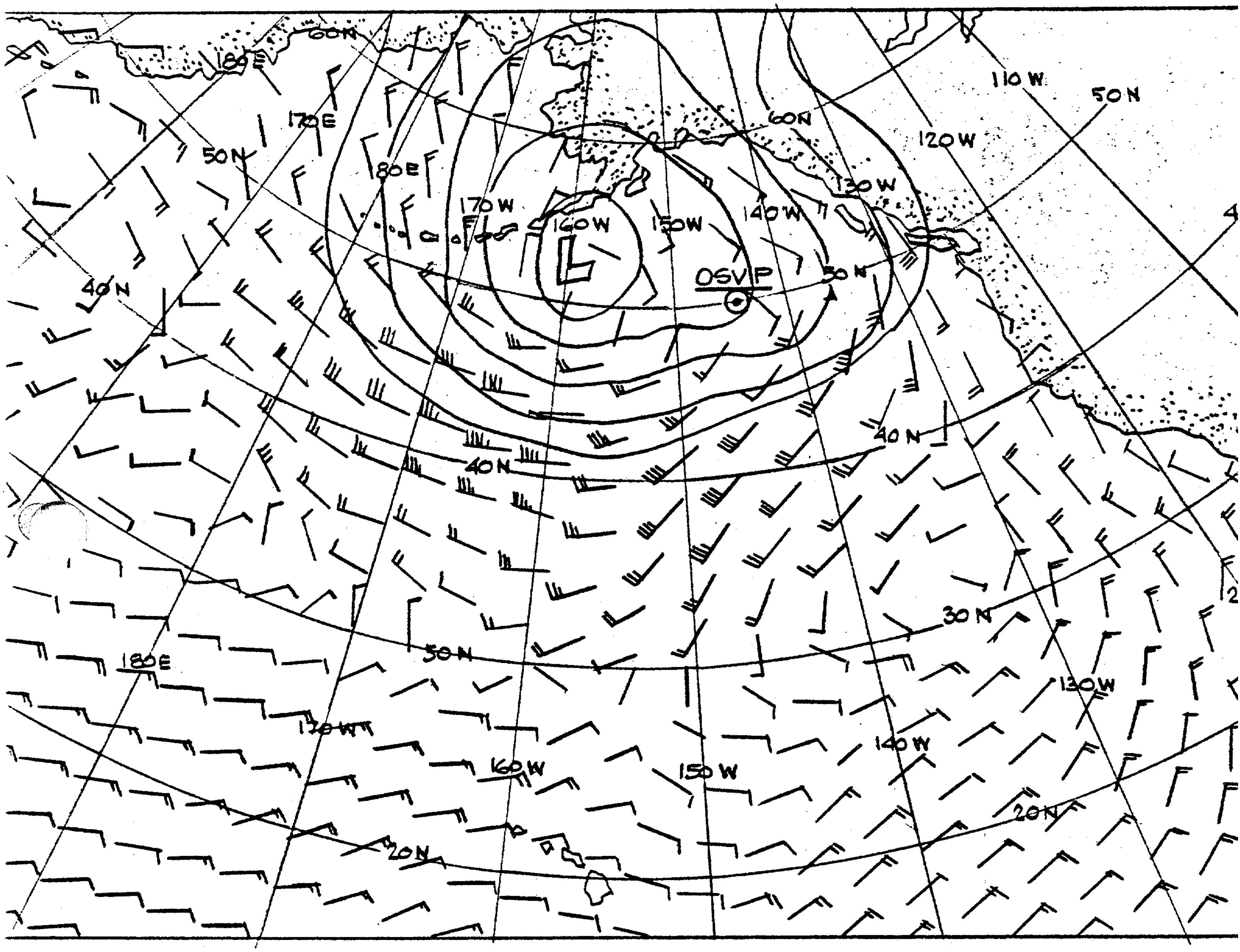
(U.S. NAVY MARINE CLIMATIC ATLAS)

FIGURE 1-14



STORM TRACES IN THE BERING SEA 1949-1975

FIGURE 1-15

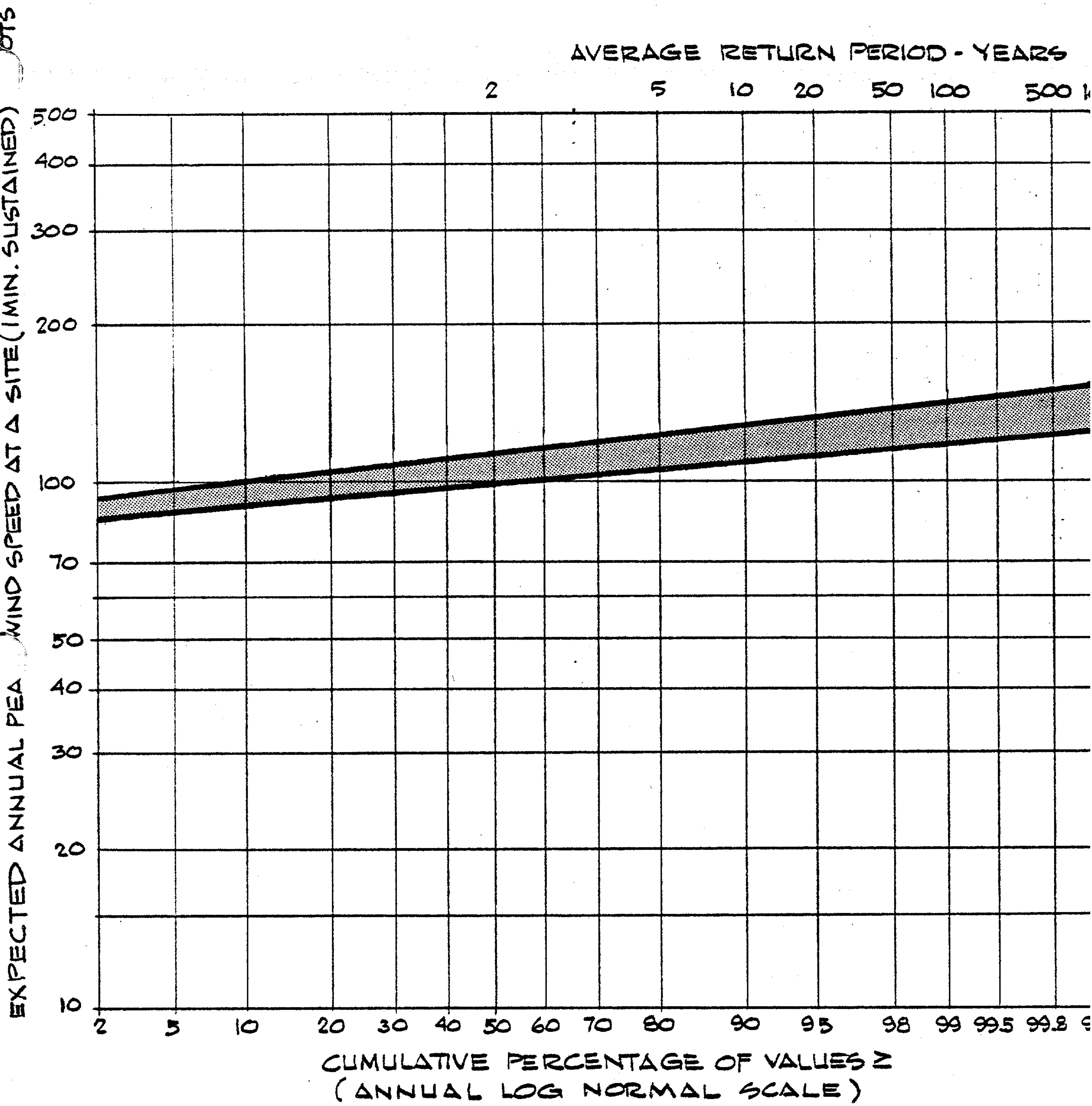


SYNOPTIC MAP

OF

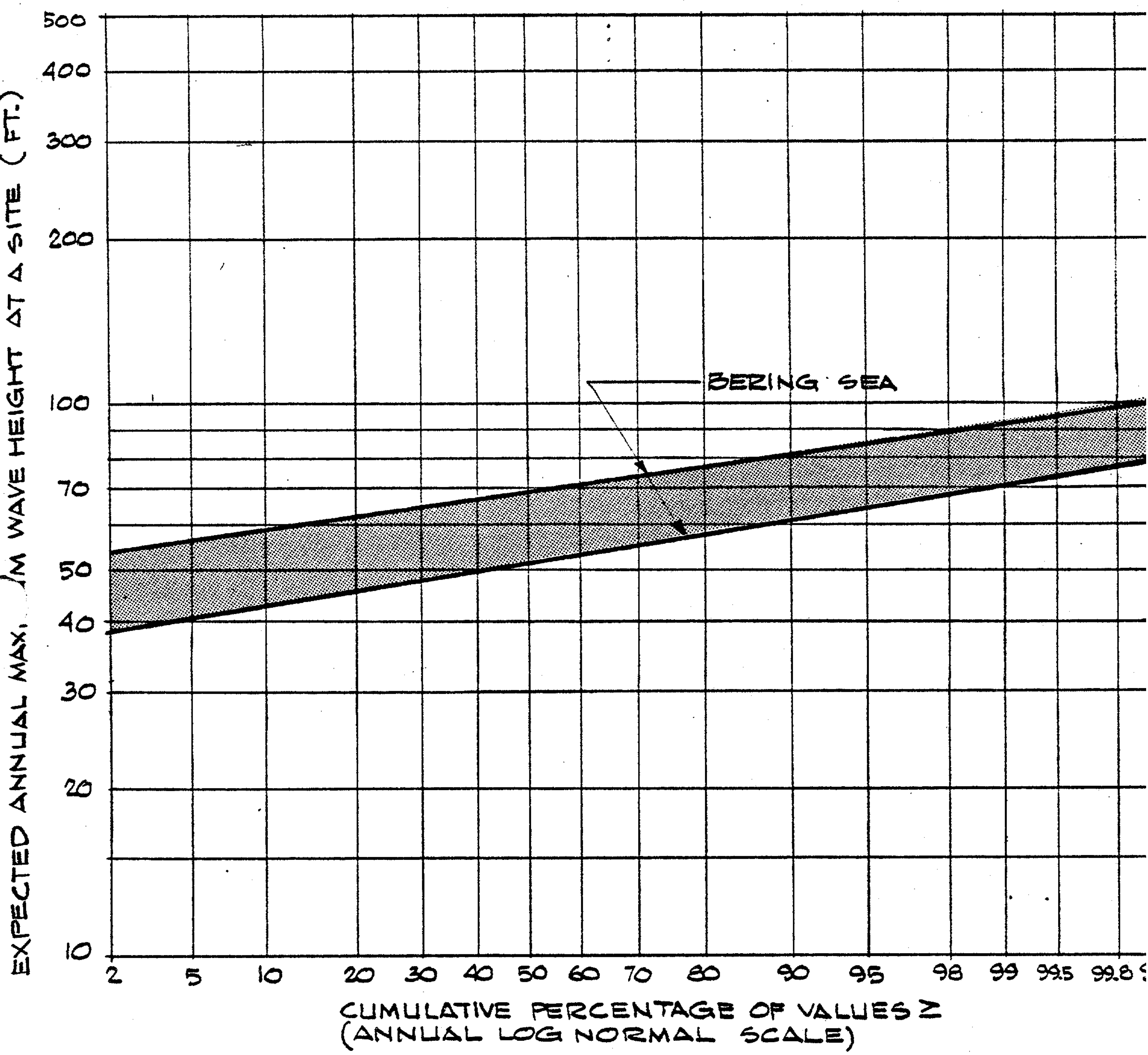
LOW PRESSURE STORM SYSTEMS IN BERING SEA

FIGURE 1-16



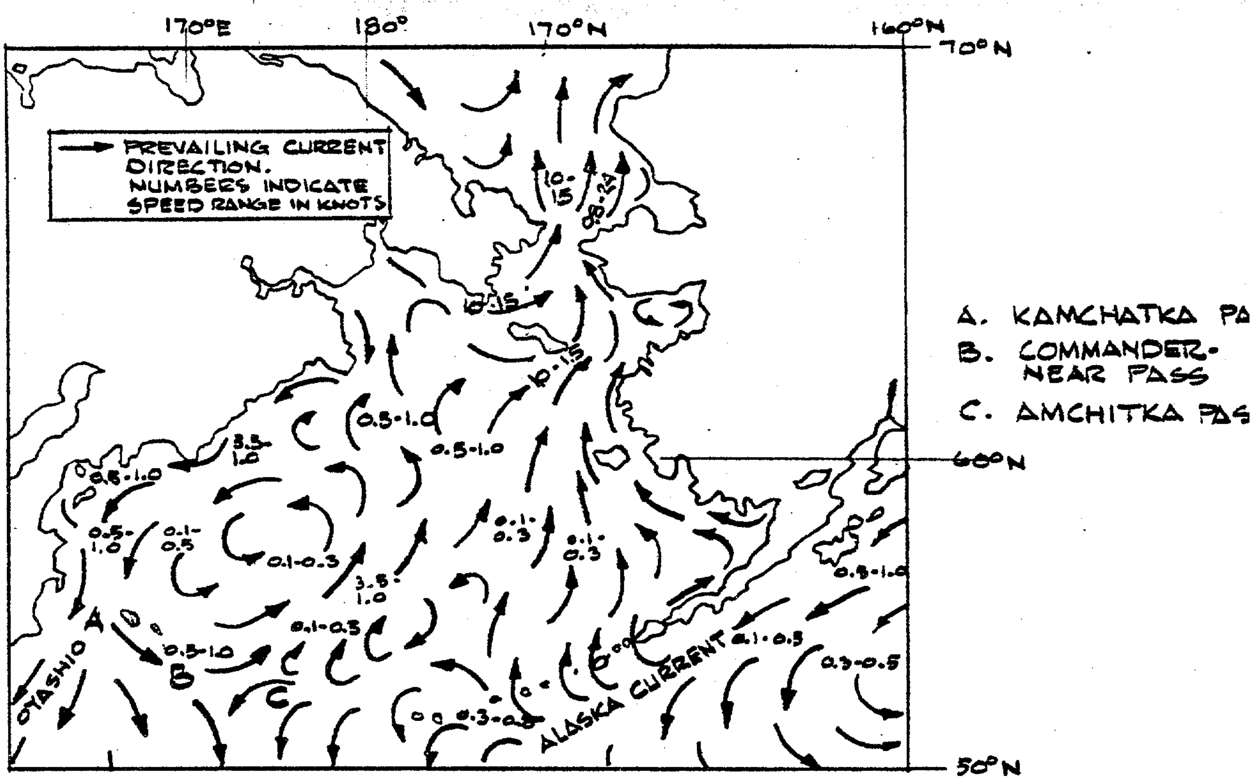
BERING SEA WIND SPEEDS

FIGURE 1-17

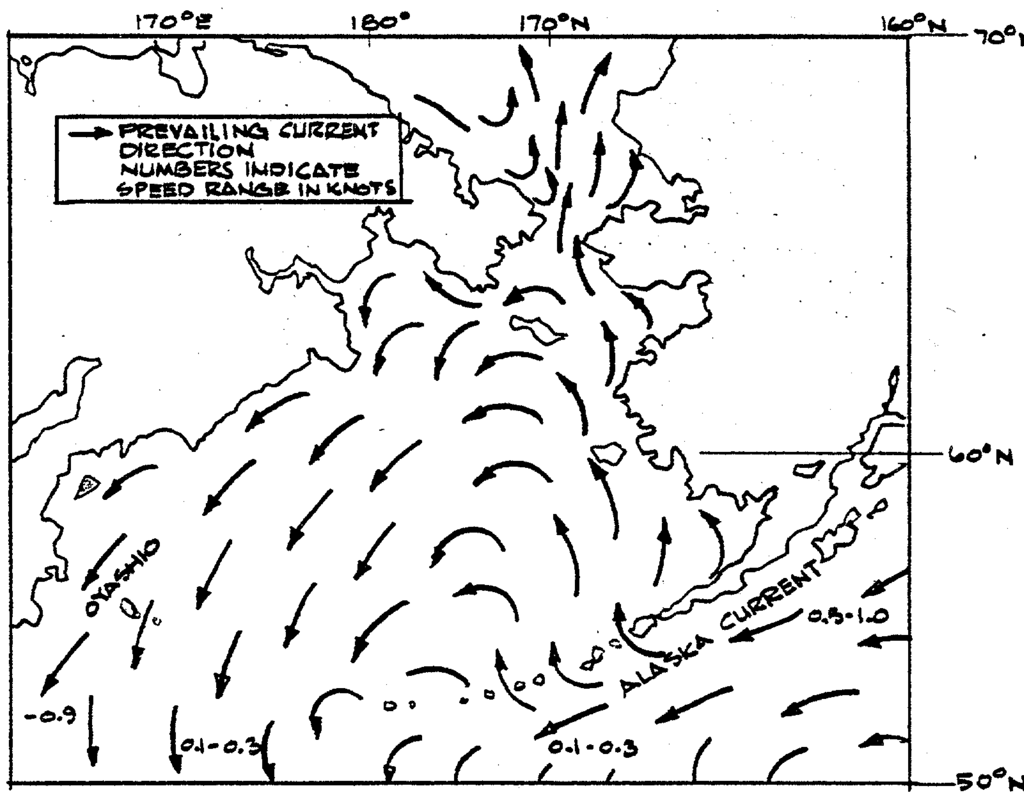


BERING SEA WAVE HEIGHTS

FIGURE 1-18



SUMMER

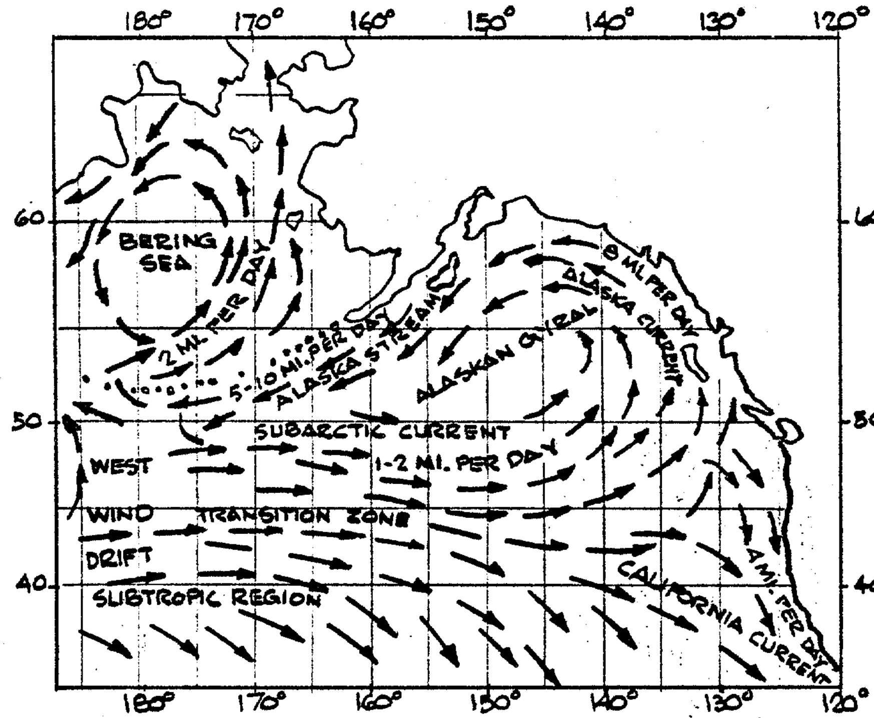


WINTER

BERING SEA SURFACE CURRENTS

(U.S. NAVY CLIMATIC ATLAS)

FIGURE 1-19



CIRCULATION CURRENTS

FIGURE 1-20

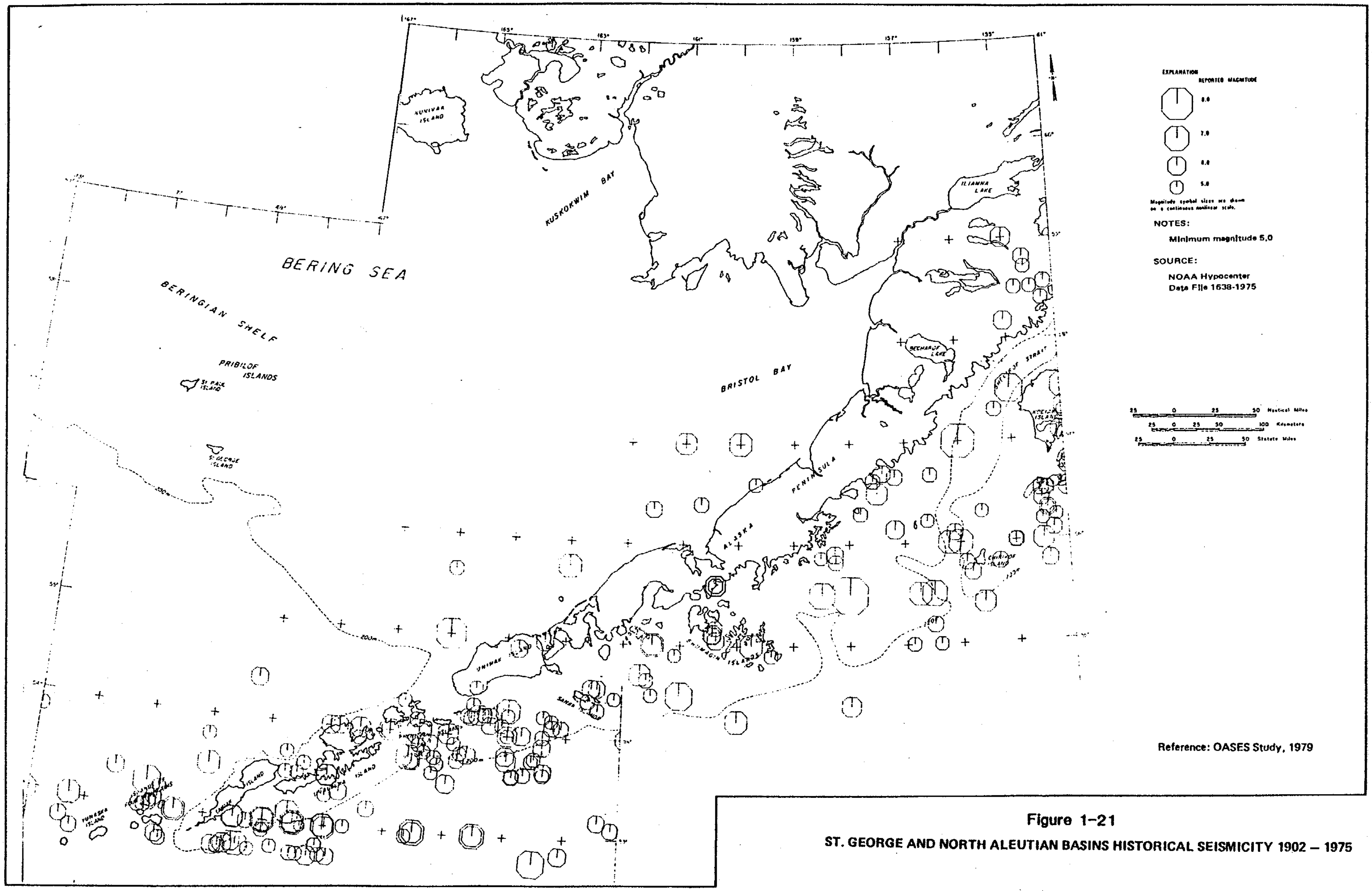
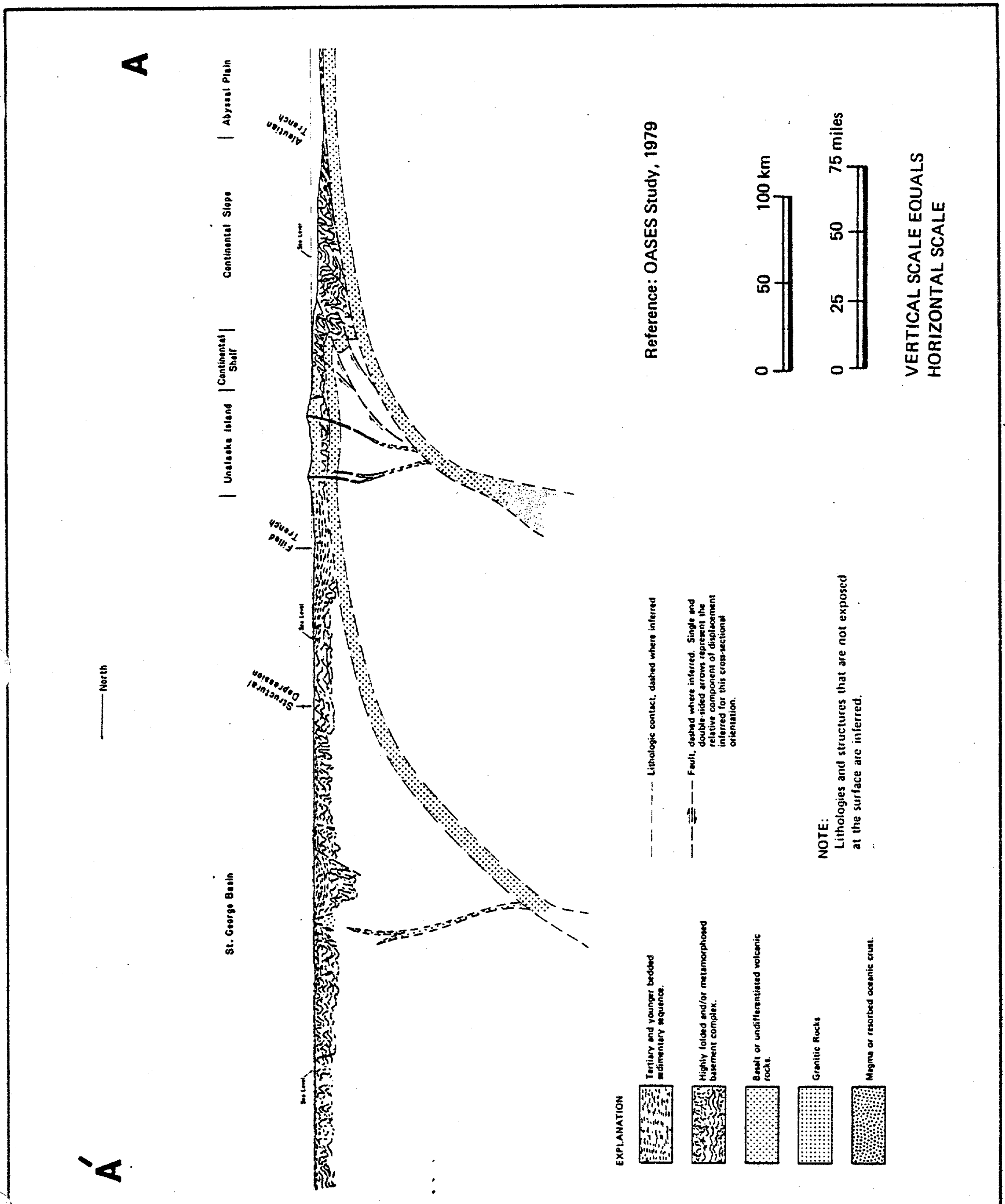


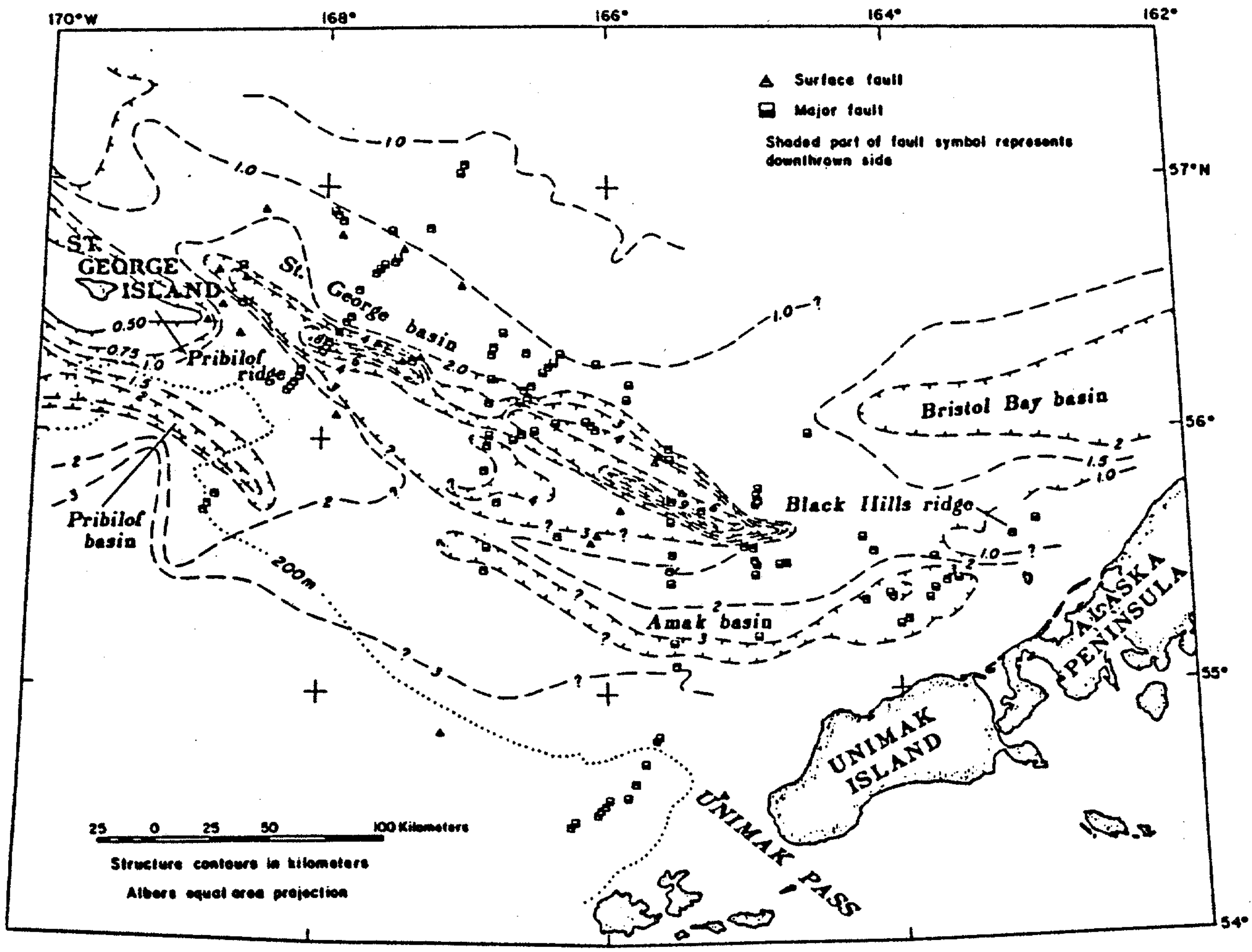
Figure 1-21

ST. GEORGE AND NORTH ALEUTIAN BASINS HISTORICAL SEISMICITY 1902 – 1975



ST. GEORGE BASIN REGION SECTION A' - A, SCHEMATIC TECTONIC MODEL

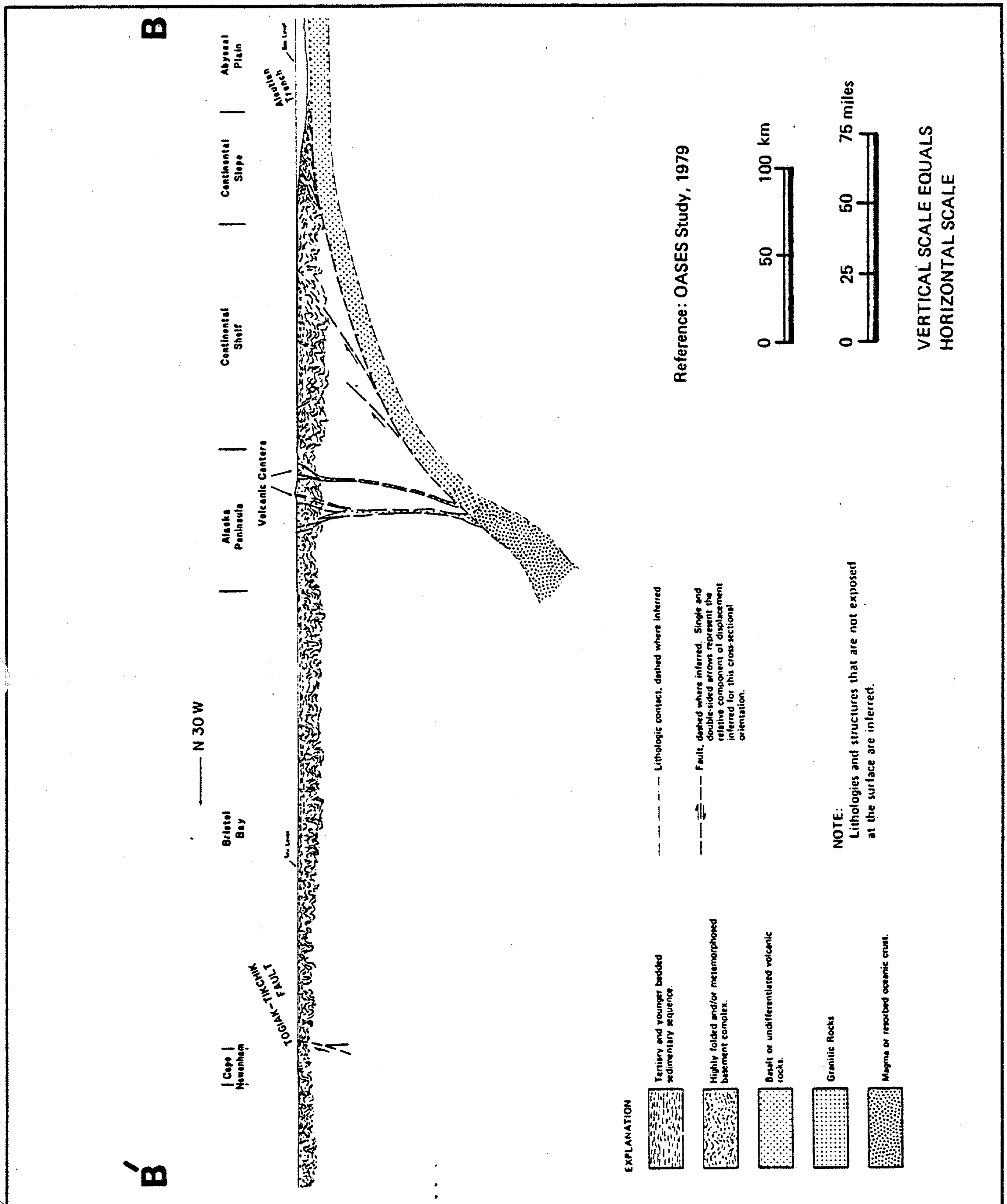
Figure 1-22



Reference: Marlow et al., 1979

DISTRIBUTION OF SURFACE AND MAJOR FAULTS

Figure 1-23



NORTH ALEUTIAN REGION SECTION B' - B.
SCHEMATIC TECTONIC MODEL

Figure 1-24

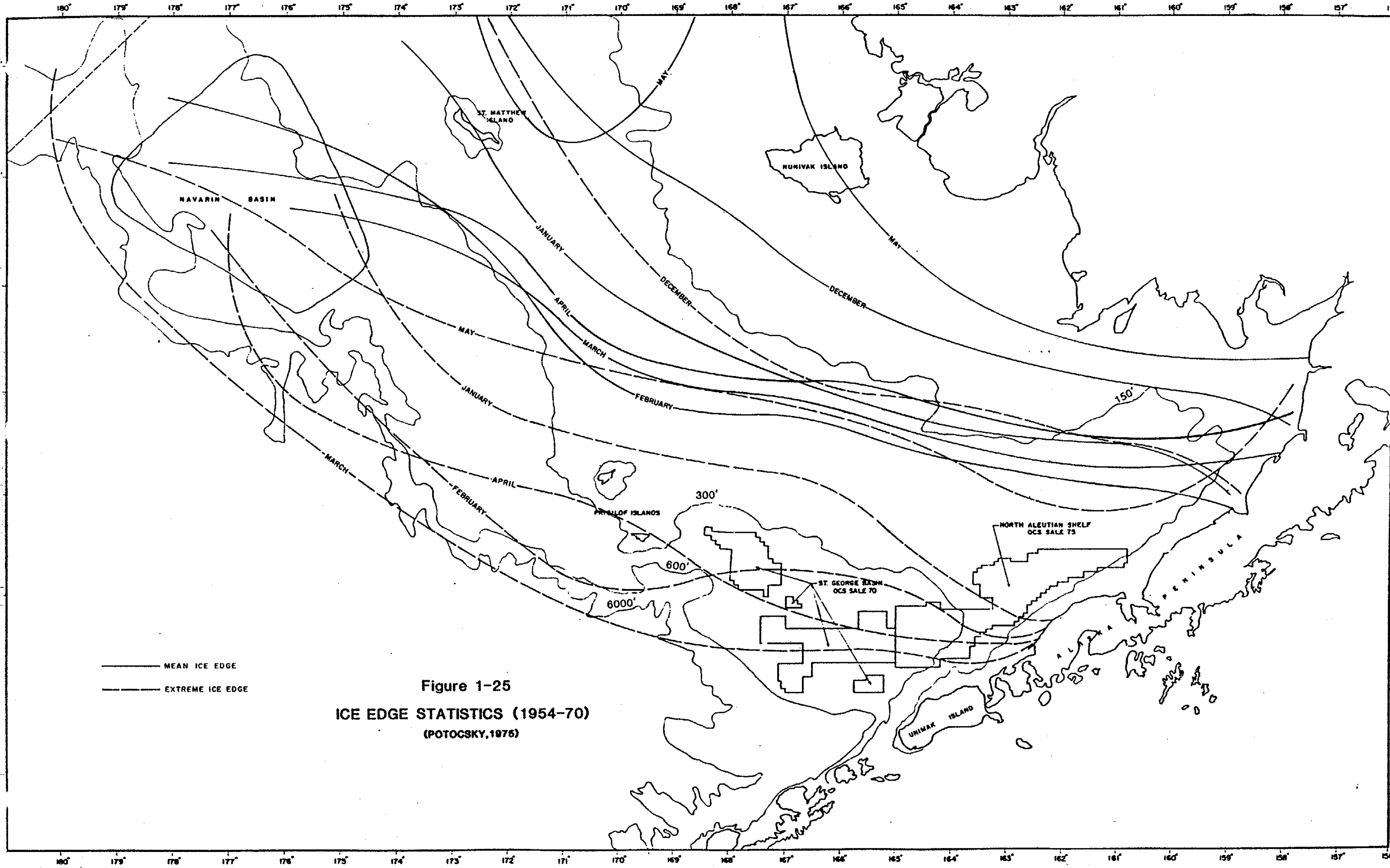
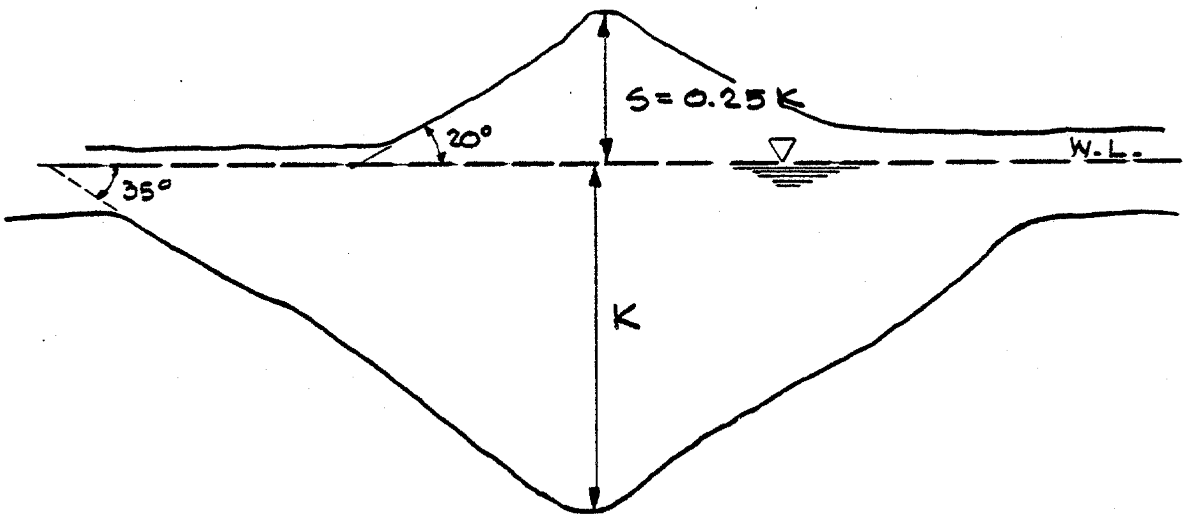
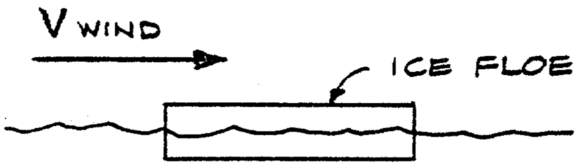


Figure 1-25
ICE EDGE STATISTICS (1954-70)
(POTOCSKY,1976)

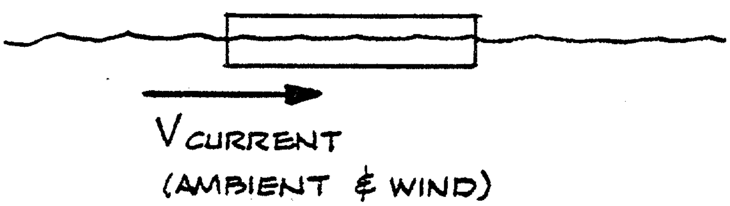


RIDGE CROSS SECTION

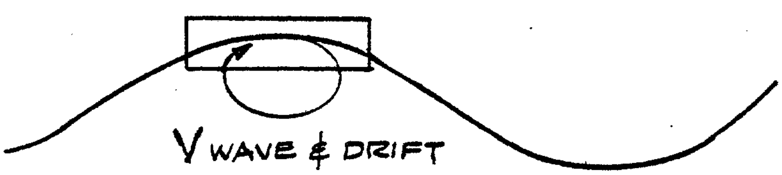
FIGURE 1-26



$$= V_{ICE/WIND}$$



$$= V_{ICE/CURRENT}$$

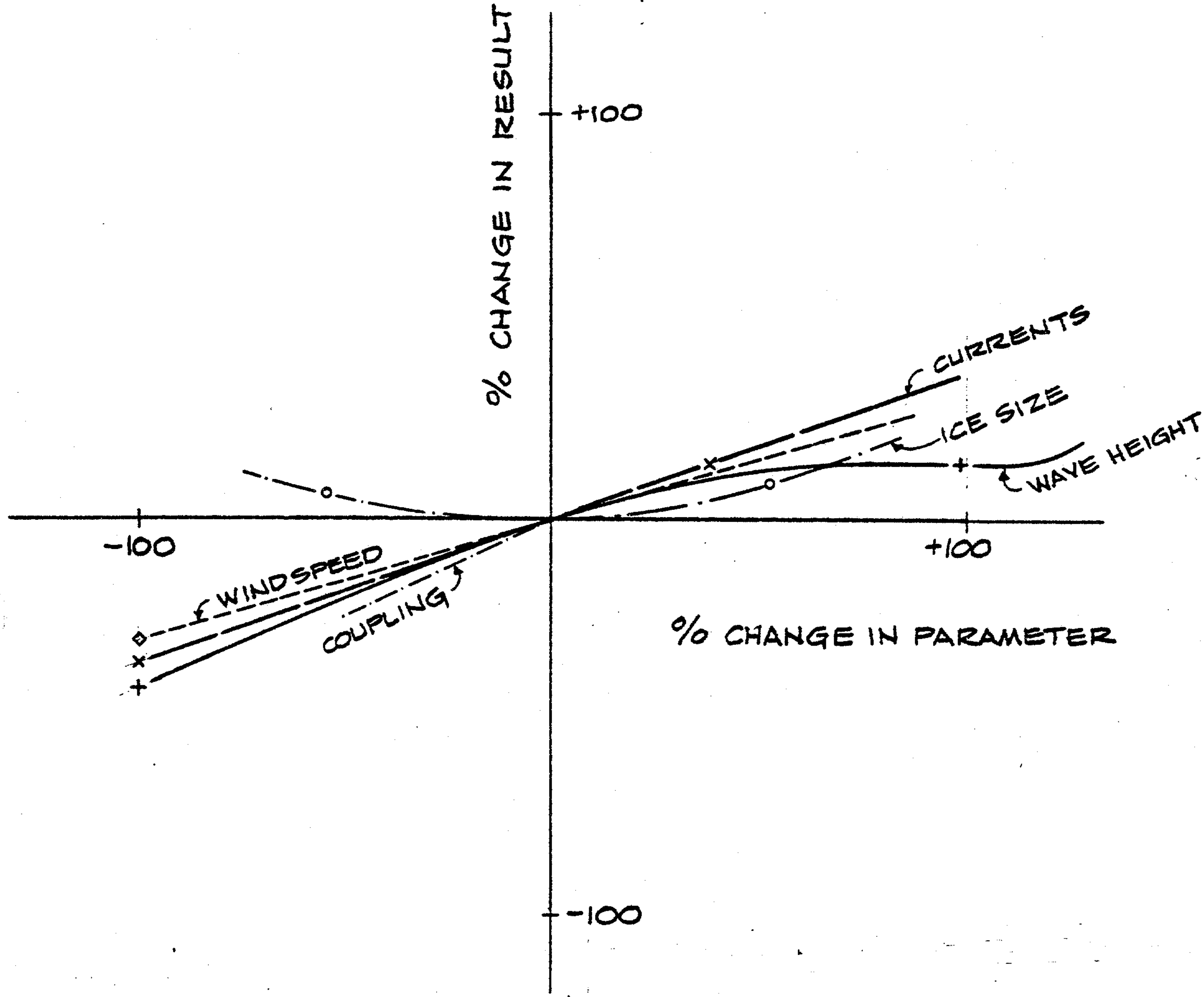


$$= V_{ICE/WAVE}$$

$$V_{ICE} = V_{ICE/WIND} + V_{ICE/CURRENT} + V_{ICE/WAVE}$$

ANALYTICAL MODEL

FIGURE 1-27



PARAMETRIC SENSITIVITY FOR RESULTANT ICE VELOCITY

FIGURE 1-28

TABLE 1-1
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: ST. GEORGE BASIN

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Oceanographic			
Water Depth (ft)	450	300	600
Wind Speed (mph)	130	120	140
Wave Height (ft)	85	75	90
Wave Period (sec)	15	14	17
Current Speed - Surface (fps)	4	0	5
Current Speed - Bottom (fps)	0	0	1
Surge and Tide Height (ft)	7	5.5	8.5
Deck Elevation (ft + MWL) (to bottom of support)(1)	64	56.5	68.5
Soil			
Soil Type A (cohesive)) Figure 1-5	Medium Clay	Soft Clay	Stiff Clay
Soil Type B (granular) Figure 1-6	Med. Sand.	Loose Sand	Dense Sand

(1)Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 1-1, CONT.

LOCATION: ST. GEORGE BASIN

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Earthquake			
Ground Acceleration (g's)	0.125	0.10	0.15
Ground Velocity (ips)	4.25	3.5	5.0
Ground Displacement (in)	0.5	0.4	1.0
Acceleration Response Spectrum	Figure 1-7	Figure 1-7	Figure 1-7
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	2	2	4
Rafted (Consolidated) Floe Thickness (ft)	6	4	7
Ridging (Including Unconsolidated Ice) Thickness (ft)	15	12	18
Rafted Ice Loading (kips/ft)	85	55	110
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	60	40	90

TABLE 1-2
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: NAVARIN BASIN

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Oceanographic			
Water Depth (ft)	450	300	600
Wind Speed (mph)	130	120	140
Wave Height (ft)	80	70	85
Wave Period (sec)	14	13	15
Current Speed - Surface (fps)	4	0	5
Current Speed - Bottom (fps)	0	0	1
Surge and Tide Height (ft)	5	4	7
Deck Elevation (ft + MWL)(1)	59	52	64
Soil			
Soil Type A (cohesive) Figure 1-5	Med. Clay	Soft Clay	Stiff Clay
Soil Type B (granular) Figure 1-6	Med. Sand	Loose Sand	Dense Sand

(1)Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 1-2, CONT.

LOCATION: NAVARIN BASIN

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Earthquake			
Ground Acceleration (g's)	0.05	0.01	0.10
Ground Velocity (ips)	1.5	1.0	2.0
Ground Displacement (in)	---	---	---
Acceleration Response Spectrum	Figure 1-8	Figure 1-8	Figure 1-8
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	4	2	5
Rafted (Consolidated) Floe Thickness (ft)	8	6	9
Ridging (Including Unconsolidated Ice) Thickness (ft)	20	15	25
Rafted Ice Loading (kips/ft)	115	80	140
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	110	60	170

TABLE 1-3
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: NORTH ALEUTIAN SHELF

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Oceanographic			
Water Depth (ft)	300	200	350
Wind Speed (mph)	120	110	130
Wave Height (ft)	78	65	85
Wave Period (sec)	15	14	16
Current Speed - Surface (fps)	5	0	6
Current Speed - Bottom (fps)	2	0	3
Surge and Tide Height (ft)	12	8	15
Deck Elevation (ft + MWL)(1)	65	53	72
Soil			
Soil Type A (cohesive) Figure 1-5	N/A	N/A	N/A
Soil Type B (granular) Figure 1-6	Med. Sand	Loose Sand	Dense Sand

(1)Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 1-[illegible], CONT.

LOCATION: NORTH ALEUTIAN SHELF

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Earthquake			
Ground Acceleration (g's)	0.24	0.18	0.3
Ground Velocity (ips)	8	6.5	10
Ground Displacement (in)	1.5	1.0	2
Acceleration Response Spectrum	Figure 1-9	Figure 1-9	Figure 1-9
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	3	1	4
Rafted (Consolidated) Floe Thickness (ft)	6	4	7
Ridging (Including Unconsolidated Ice) Thickness (ft)	15	12	18
Rafted Ice Loading (kips/ft)	85	55	110
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	60	40	90

TABLE 1-4
SUMMARY OF PRINCIPAL INFORMATION ON ENVIRONMENTAL CONDITIONS NEEDED TO ALLOW DEVELOPMENT OF BERING SEA BASINS WITHOUT UNDUE DELAYS OR COSTS

CATEGORY	EFFORT	JUSTIFICATION
Soils	Site specific soil borings	Design mat and pile foundations for platforms.
	Route specific soil surveys	Design pipelines and select optimum routes.
	Geohazard and bathymetric surveys and studies for platform sites	Assess presence and nature of geohazards and bathymetry to allow optimum siting and accommodating design strategies.
	Geohazard and bathymetric surveys and studies for platform routes	Assess presence and nature of geohazards and bathymetry to allow optimum siting and accommodating design strategies.
	Scour potential evaluation	Based on geohazard surveys (regional scour data), data on currents and waves, platform and pipeline configurations define potential for local and regional scour important to foundation designs.

TABLE 1-4, CONT.

CATEGORY	EFFORT	JUSTIFICATION
Earthquake	Local geologic and site effects	Surveys, measurements, and analytical data to indicate influence on ground motions.
	Source surveys and assessments	Surveys, measurements, and analytical data to indicate locations and characteristics of significant earthquake potential producing sources and their transmission (attenuation) characteristics.
	Seismic exposure evaluations	Surveys, measurements, and analytical data to indicate site and route specific seismic exposure (strong ground motions) appropriate for design of specific production systems.

TABLE 1-4, CONT.

CATEGORY	EFFORT	JUSTIFICATION
Ice	Ice measurements	Surveys, field tests, and laboratory studies intended to identify ice growth and strength characteristics.
	Ice features	Surveys, field tests, and laboratory studies intended to identify nature, size and strength characteristics of ice features.
	Design conditions	Based on hindcast studies and data analyses, develop reliable evaluations of intense ice conditions and forces appropriate for specific areas and production systems.
	Operating conditions	Based on hindcast studies and data analyses, develop reliable evaluations of operating ice conditions and force appropriate for specific locales, types of operations, and production systems.
	Wave ice interactions	Surveys, field observations, and analytical model data, accurately evaluate design force and operating condition strategies appropriate for interactions between waves, ice-edge breakup features, and specific production systems and components.
	Ice flexible, multileg	Based on surveys, field observations, laboratory studies, structure interactions and results from analytical models, assess the design conditions appropriate for interactions of multi-leg, flexible structures with ice fields.

TABLE 1-4, CONT.

CATEGORY	EFFORT	JUSTIFICATION
Design Criteria	Environmental	Recognizing the multi-hazard characteristics of the Bering Sea area, and based on the realistic characterization of the oceanographic, soil, earthquake, and ice conditions-forces, and on the structural-foundation criteria, economic and safety considerations associated with specific production systems, develop appropriate design basis environmental conditions.
	Structural	Environmental, structural, and foundation criteria are inter-related through economic and safety requirements placed on specific production systems. Develop design basis structural criteria based on these recognized interactions and considerations.
	Foundation	Adequate performance, economy, and safety are required of foundation elements and the supporting soils. Recognizing the specific interactions and considerations associated with specific production systems, develop appropriate design basis foundation design criteria for proposed system sites and routes.

TABLE 1-4, CONT.

CATEGORY	EFFORT	JUSTIFICATION
Oceanography	Wave measurements	Provide sufficient data to calibrate hindcasting models and permit assessment of appropriate design wave conditions and operational conditions.
	Current measurements	Provide sufficient data to calibrate hindcasting models and permit assessment of appropriate design current conditions and operational conditions.
	Wind measurements	Provide sufficient data to calibrate hindcasting models and permit assessment of appropriate design wind conditions and operational conditions.
	Design conditions	Based on hindcast studies and data analyses, develop accurate characterizations of extreme oceanographic conditions (wind, wave, current) and forces appropriate for specific areas and production systems.
	Operating conditions	Based on hindcast studies and data analyses, develop reliable characterizations of operating oceanographic conditions appropriate for specific areas, types of permutations, and production systems.

TABLE 1-5
WAVE-ICE INTERACTION ANALYSIS PARAMETERS

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
CASE NO.	ICE LENGTH m	ICE WIDTH m	ICE THICK-NESS m	SHAPE CATEGORY	WATER DEPTH m	WIND VELOC. kn	SURFACE CURRENT LAYER DEPTH m	LATITUDE °N	COUPLING COEFF.	AMBIENT CURRENT m/s	WIND CURRENT m/s	WIND CURRENT DEPTH m	CURRENT CODE	WAVE CODE	WAVE HEIGHT m	WAVE PERIOD sec.
1	20	20	3	5	150	100	10	60	1.0	0.5	1.0	10	2	2	5	10
2	20	20	3	5	150	100	10	60	1.0	0.0	0.0	10	2	2	5	10
3	20	20	3	5	150	100	10	60	1.0	0.5	1.0	10	2	2	0	0
4	20	20	3	5	150	0	10	60	1.0	0.5	1.0	10	2	2	5	10
5	20	20	3	5	150	100	10	60	1.0	0.0	1.7	72	1	1	35.4	20.9
6	20	20	3	5	150	50	10	60	1.0	0.0	0.54	46	1	1	14.4	13.5
7	20	20	3	3	150	100	10	60	1.0	0.5	1.0	10	2	2	5	10
8	20	20	3	5	150	100	10	60	1.0	0.5	1.0	10	2	2	10	15
9	20	20	3	5	150	100	10	60	1.0	0.5	1.5	10	2	2	5	10
10	20	20	3	5	150	100	10	60	0.5	0.25	0.5	10	2	2	5	10
11	20	20	3	5	150	100	10	60	0.8	0.4	0.8	10	2	2	5	10
12	10	10	3	5	150	100	10	60	1.0	0.5	1.0	10	2	2	5	10
13	10	10	3	5	150	100	10	60	1.0	0.5	1.0	10	2	2	15	12
14	30	30	4	2	150	100	10	60	1.0	0.5	1.0	10	2	2	5	10
15	30	30	4	2	150	100	10	60	1.0	0.0	0.0	10	2	2	5	10
16	30	30	4	2	150	100	10	60	1.0	0.5	1.0	10	2	2	0	0
17	30	30	4	2	150	0	10	60	1.0	0.5	1.0	10	2	2	5	10
18	30	30	4	2	150	100	10	60	1.0	0.5	1.7	72	1	1	35.4	20.9
19	30	30	4	2	150	50	10	60	1.0	0.5	0.54	46	1	1	14.4	13.5
20	20	20	3	5	150	50	10	60	1.0	0.0	0.0	10	2	2	5	10
21	20	20	3	5	150	10	10	60	1.0	0.0	0.0	10	2	2	5	10
22	20	20	3	5	150	100	10	60	1.0	0.5	2.0	10	2	2	5	10
23	20	20	3	5	150	50	10	60	1.0	0.5	2.0	10	2	2	5	10
24	20	20	3	5	150	10	10	60	1.0	0.5	2.0	10	2	2	5	10
25	20	20	3	5	150	112.8	10	60	1.0	0.2	1.0	10	2	2	24.4	14
26	20	20	3	5	150	104.2	10	60	1.0	0.0	0.0	10	2	2	21.4	13
27	20	20	3	5	150	121.5	10	60	1.0	0.5	1.0	10	2	2	25.9	15

TABLE 1-6
DEFINITIONS OF COLUMNS FOR TABLE 1-5

The columns of input data (Table 1-5) are as follows:

1. Iceberg length, m.

2. Iceberg width, m.

3. Iceberg thickness, m. See item 4 below for comment on appropriate thickness for certain shaped icebergs.

4. Shape category. Specify as follows:

Tabular-smooth	1
Tabular-rough	2
Blocky	3
Pinnacle/drydock	4
Floe (sea ice)	5

   For aid in determining the thickness to be used for 3. above, the height of the iceberg can be multiplied by the appropriate ratio to give thickness. For a tabular smooth, the ratio is 7.0/1., tabular rough is 6.0/1., blocky is 5.26/1., pinnacle/drydock is 3.35/1., and floe ice is 7.0/1.

5. Water depth, m.

6. Wind velocity, knots.

7. Initial wind driven current layer depth, m., used to develop final mixed layer depth within the program.

8. Latitude.

9. Desired coupling coefficient between any moving water current and the iceberg. 1.00 means 100% coupling.

10. Ambient tidal or sea current as desired, m/s.

11. Operator specified wind driven current velocity within the specified mixed layer of 7. above, m/s.

12. Operator specified mixed layer depth for wind driven current, m.

13. Operator's instructions on data use choice:

    1 - means use given wind to develop wind driven current and mixed layer depth.

    2 - means use operator <u>specified</u> wind driven current <u>and</u> mixed layer depth.

14. Operator's instructions on data use choice:

    1 - means use given wind to develop all wave data.

    2 - means use operator specified wave height and period data.

15. Operator specified wave height, m.

16. Operator specified wave period, seconds.

    Note: If operator specified wave height and period are used, the program assumes the height to be the average of the highest 10% of the waves and therefore can also compute a significant wave height value for use in the wave reflection force calculation.

TABLE 1-7

INITIAL SENSITIVITY STUDY
RESULTS

CASE	INPUT CHANGE	RESULT (fps)
1	Base case	13.8
2	Omit currents	8.9
3	Omit waves	7.9
4	Omit wind	9.4
5	Compute currents and waves	28.1
6	Compute currents and waves for a wind of 50 knots	15.1
7	Change shape from floe (5) to blocky (3)	14.3
8	Wave height 10 m Wave period 15 s	15.8
9	Wind current 1.5 m/s	15.5
10	Coupling coeff. 0.5 Ambient current 0.25 m/s Wind current 0.5 m/s	10.6
11	Coupling coeff. 0.8 Ambient current 0.4 m/s Wind current 0.8 m/s	12.6
12	Ice length and width 10 m	13.6
13	Ice length and width 10 m Wave height and period 15 m, 12 sec	21.1
14	Ice length and width 30 m	16.2
15	Ice length and width 30 m Omit current	11.2
16	Ice length and width 30 m Omit wave	10.6

TABLE 1-7, CONT.

CASE	INPUT CHANGE	RESULT (fps)
17	Ice length and width 30 m Omit wind	9.0
18	Ice length and width 30 m Compute current and wave	32.2
19	Ice length and width 30 m Compute current and wave Wind speed 50 knots	17.7
20	Wind of 50 knots Omit currents	7.4
21	Wind of 10 knots Omit currents	4.8
22	Wind current 2.0 m/sec	17.1
23	Wind of 50 knots Wind current 2.0 m/sec	15.6
24	Wind of 10 knots Wind current 2.0 m/sec	13.0
25	Wind of 112.8 knots Ambient current 0.2 m/sec Wind current 1.0 m/sec Wave height 24.4 m Wave period 14 sec	25.7
26	Wind of 104.2 knots Wave height 21.4 m Wave period 13 sec Omit currents	20.5
27	Wind of 121.5 knots Wave height 25.9 Wave period 15 sec	26.9

TABLE OF CONTENTS

SECTION 2.0

LIST OF FIGURES

SECTION 2.0

2.0 TOPSIDE DRILLING AND PRODUCTION FACILITIES

2.1 Scope of Work

The scope of work for conceptual design of the topside facilities on the offshore platform alternatives consisted of the following elements:

- For two cases of crude oil production rates, the topside facilities were to be sized and adapted to the severe environmental conditions existing in the Bering Sea.

- Two alternative support structures were to be considered -

  - a single legged, central caisson or monotower
  - a 4-legged structure

The monotower was to be a concrete structure with crude oil storage in the base of the structure. This would require a utility shaft sharing the available space inside the single leg with the well conductors and other risers. It should be noted that the monotower was found not to be a favored structural alternative and that work on monotower topside facilities was not carried through in as much detail as the 4-legged deck.

The 4-legged structure would be either a steel jacket or a concrete structure. In the former case, no base storage would be provided. In both cases, drilling would be done through two of the four legs leaving the other two legs available for utility water and firewater intakes.

- Crude oil will be delivered to a shore terminal by submarine pipeline or stored offshore for direct tanker loading from an offshore terminal. The separate requirements for the topside facilities in these transportation alternatives were considered. The principal differences in topside requirements are:

-- Crude oil must be stabilized to one atmosphere for local storage at the platform, and treatment facilities must be provided for the subsea tankage ballast water.

-- Mainline pumps are required for pipelining crude to shore.

- The properties of the crude oil in each of the three lease areas to be examined, North Aleutian, St. George and Navarin, are not known. Assumptions were made as necessary to facilitate the conceptual design. The assumptions and criteria adopted are given in the next section.

- Layouts of the topside components were to be made which would aid in the description of the conceptualized systems and in the development of the order of magnitude cost estimation.

- Total topside operating weights were to be estimated for each case of production to assist in platform structure design.

- Order of magnitude capital costs were to be developed for two cases of crude oil production rate.

2.2 Design Criteria and Assumptions

- The two cases of crude oil production rates to be considered were 100 thousand barrels per day (MBPD)and 300 MBPD.

- The gas-oil ratio (GOR) was taken to be 1000 standard cubic feet per barrel.

- Gas would be re-injected into the reservoir.

- Treated water would be injected into the reservoir in the proportion of one barrel of water for one barrel of oil produced.

- In the offshore storage and tanker loading option, the oil would be stabilized for tanker transport. The volatiles would be used as fuel on the platform or injected into the reservoir.

- In the offshore storage option, produced water and oily water treatment would be required only for the storage tank ballast water. No provision would be made for tanker ballast water on the assumption that the tankers would be dedicated and will not require ballast water discharge at the loading terminal.

- In the onshore crude oil storage option with pipeline transportation from the platform to shore, no ballast water treatment at the platform would be required. Space and weight allowance allotted for ballast water treatment in the offshore loading option would be used for mainline crude oil pumps.

- A total of 30 well slots was assumed for the 100 MBPD production case and a total of 48 well slots for the 300 MBPD production case.

- All well casings, conductors and risers would be contained within the structure legs to shield these members against ice loading.

- No floating accommodation facilities (floatels) were planned for hookup and drilling operations, since these activities would continue during conditions of winter sea ice. All personnel would be quartered on the platform. The maximum number of beds would be 360 for the 100 MBPD production case and 432 for the 300 MBPD case. Still, larger staffing could be provided to implement hookup work by adding additional quarters modules. Each module accommodates 72 persons. Normal staff during production operations were assumed to be 200 persons and 350 persons for the respective cases of throughput. All beds would be utilized for hookup and hookup support staff until hookup is virtually complete. Drilling operations with two rigs were estimated to require a minimum of 70 persons.

- All living and working spaces would be enclosed to minimize ice accumulation and to provide protection from the weather.

- Since platform re-supply could be delayed significantly during the winter season, an additional weight allowance was made for bulk and liquid storage on the platform. This allowance was assigned a minimum of 1000 short tons for dry bulk and the same weight allowance for liquid storage for both production cases.

- Topside components would be modularized whether assembled offshore at the site of production or nearshore with a subsequent tow-out of the completed platform. The maximum module lift weight was assumed to be 1600 tons.

- For topsides on a steel structure, all hookup would be done offshore.

- For topsides on a concrete gravity base, 90% of the topsides hookup would be done nearshore at a shipyard. The remaining 10% of the hookup work would be done offshore.

- For topsides on a concrete gravity base with subsea storage and a utility shaft, a manhour allowance is given for interfacing the topside and utility shaft components at a nearshore, remote location.

- A manhour allowance for work offshore is made in the case of gravity structures for preparing the platform for startup.

- For reasons of personnel safety, living quarters were placed on the opposite side of the platform from the hydrocarbon production and treatment facilities.

- Crude oil properties assumed:

  - API Gravity: 25° to 35°
  - pour point temperature less than 35°F
  - water cut: 10%
  - total sulfur: negligible

2.3 Well Arrangements

The topside facility layouts are essentially dependent on the choice of the platform structure; and, in particular, on the choice of a monotower or of a structure with two or more legs.

For reasons of equilibrium in the deck structure, a monotower platform is supported at the center of the deck. To keep all well slots within this single leg, drilling must be done from the center of the platform, and production risers must share the available space with the utility shaft and all other risers. This situation results in crowded conditions for all components in the single leg; but, most importantly, only one drilling derrick can be installed. Interference between rigs would make it very difficult to operate more than one.

At the beginning of this study, the monotower structure was accepted as a viable alternative in the expectation that ice bridging and the resulting loads on multi-legged structures might be excessive. Analysis of the matter later showed this not to be the case as discussed elsewhere in this report. The monotower arrangement was eliminated from further consideration. However, analysis of well slot arrangements and topside layouts had been completed by that stage of the work. These results are included here for information in comparison with arrangements developed for multi-legged structures, but costs are developed only for the multi-leg.

2.3.1 Monotower Well Arrangements

The well layouts for the 100 MBPD and the 300 MBPD production cases are shown in Figures 2 - 1 and 2 - 2. The inside diameter of the monotower is taken as 74 feet for both production cases.

Only the alternative of crude oil storage in the base of the structure was considered for the concrete monotower, and nearly half of the available area in the leg is taken up by the utility shaft. This is shown as a cross section developed as two overlapping circles.

The fewer number of well slots in the case of 100 MBPD production allows a square matrix array of slots measuring 7-1/2 feet center to center. This gives adequate access to the wellheads, but crowding is further lessened by locating alternate rows of wellheads at a different level. The wellheads marked in black in Figure 2 - 1 are at one level separated by a vertical distance of about 25 feet from the wellheads shown as open circles.

Placement of 48 wellheads within the same available space leads to a much denser spacing. As shown in Figure 2 - 2, the wellheads are spaced along the perimeter of three circles. Again, two levels are used for the wellheads as marked by black and open circles.

Conductor casing diameters are taken arbitrarily to be 24 inches in all cases and the center-to-center spacing of the casings is more than 60 inches in the circular arrays. Seemingly tight, this still exceeds wellhead spacings as small as 27 inches that have been used in the Gulf of Mexico for mudslide area platforms with up to 15 wells per leg.

2.3.2 Multi-Leg Well Arrangements

Four-legged structures, whether of steel or concrete, are utilized in the multi-leg category. Two vertical legs at one end of the platform would be used for well slots. Since quarters and utilities are placed at the other end of the platform structure, the remaining pair of legs would not be used for risers in hydrocarbon service.

The well legs need to be a minimum inside diameter of 15 feet for the 30-well case and 22 feet for the 48-well case to give well spacings of 29 inches. Two derricks can be accommodated - one for each leg. Structural considerations may result in greater leg size; however, layouts are shown on this minimum spacing basis.

Once all wells have been completed, one derrick can be removed from the platform leaving one for well workover in either leg. The well slots are divided equally between the production legs - 15 in each for the 100 MBPD case and 24 in each for the 300 MBPD case. The well-head arrays are contained within a 15 foot I.D. circle in both cases. Although the larger case of throughput has an inside diameter of 22 feet for each well leg, the additional 3-1/2 feet of available radius has little effect on the well-head spacing.

Circular wellhead arrays and two wellhead levels are used in both cases of throughput where again, black and open circles represent wellheads at two different levels (Figures 2-3 and 2-4). Customarily the inner circle of wellheads is located at the top level.

It is evident from the spacing and dimensions shown in Figures 2-3 and 2-4 that up to 24 single-completion well slots in a 15 foot I.D. leg can be accommodated without restricting access to the wellheads. A requirement for more than 48 well slots distributed between two production legs could result in the consideration of alternative layouts. However, 48 well slots appear to be a practical maximum for a single production platform. The use of two legs for drilling instead of one as in the case of the monotower would seem to allow space for additional well conductors. The spacing of conductors within the structure legs can be controlled by conductor guides. Control of the wells' spread below seabed level is the critical factor. The center-to-center spacing of the two drilling legs of the multi-legged structure is insignificant compared to the spread of the well holes at the reservoir level.

2.4 Monotower Deck Arrangement

The monotower deck is almost square at 160 feet wide by 190 feet long for both cases of production rate and it is centered over the single concrete support leg (Figures 2-5 and 2-6).

The distribution of well slots and the location of the utility shafts as shown in Figures 2-1 and 2-2 are designed to maximize the separation between quarters and hydrocarbon production areas within the constraints imposed by a single, centrally-located platform leg.

Modules are set on two levels with separation, gas compression and water injection modules on the first level at the production end of the platform next to the flare boom. On the same level, the ballast water treatment module is located next to the quarters with the power generation and control modules on the helideck side of the platform (not shown in the layouts). Ballast water treatment is provided, since the concrete monotower is presumed to have crude storage in the base of the structure as mentioned earlier. Oil drawn down by offshore tanker loading is replaced in the tanks by seawater, which must be treated before it is discharged overboard or injected into the reservoir as newly-produced crude is delivered to the tanks. If oil storage is provided at a separate tanker loading structure, the ballast water treatment facilities will be located there and not on the production platform. This causes differences in disposal procedures as will be discussed in Section 2.6.3.

Mud, cement and storage modules occupy the second level with the single derrick, wellhead module and pipe rack module on top.

2.5 Multi-Leg Deck Arrangement

Layouts for the 4-legged structures are shown in Figures 2-7 and 2-8 . The support structure may be either steel or concrete. If steel, no storage for crude is provided in the base of the structure. Oil will be pumped to a nearby storage and tanker loading structure or through a submarine pipeline to a shore facility. For this reason, a mainline pump module is shown in the layouts with the understanding that it will be replaced by a ballast water treatment module in the case of a concrete platform with subsea storage. In such a case, further modification in these 4-legged structure layouts would be needed. Access would be required to one of the two support legs beneath the quarters-module assemblage for use as a dry utility shaft. Such modifications would not affect the general distribution of modules shown in the layouts. However, to avoid confusion with the requirements for a concrete support structure having seabed storage tanks, these layouts are restricted to a steel jacket support structure with no crude oil storage capacity.

With 4 legs for support, the deck area can be made larger than in the monotower case with greater elongation in the rectangular shape. For both the 100 MBPD and 300 MBPD cases, the deck area measures 160 feet by 220 feet. The additional space and additional operating topside weight is needed to support the operation of two drilling rigs.

The disposition of modules is much the same as in the monotower cases with drilling, production facilities and flare at the opposite end from quarters and utilities. Storage modules are in the center of the upper deck, and the pipe racks lie adjacent to the derricks on the top deck.

2.6 Description of Modules

2.6.1 Separation Module

Gas/oil separation requirements are different depending on whether the oil is to be pipelined to shore or stored offshore for direct loading to a tanker. The flow diagrams for the two cases at 300 MBPD production rate with two trains are shown in Figures 2-9 and 2-10 . Only one train of separation would be required at 100 MBPD.

For offshore crude storage, the oil must be stabilized for tanker transport, and two stages of gas oil separation are prescribed (Figure 2-9).

For transport to shore, complete stabilization is not required. There is advantage to transporting gas and gas liquids to shore where the gas could be needed for fuel and the liquids would be chilled for storage and further transportation in LPG tankers. This option shown in Figure 2-9 , has only one stage of separation with the volatile components used for power-generation, fuel and water de-oxygenation. The excess is injected.

The layout of the separation module is shown only for the situation requiring full stabilization of the crude oil (Figure 2-11). One separation train is required for the 100 MBPD case and a second train contained in a mezzanine is needed for for the 300 MBPD case. Water recovered from the crude is routed directly to the water treatment pumps without further treatment.

2.6.2 Gas Injection Module

Gas is injected into the reservoir using centrifugal gas compressors driven by gas turbines. Three stages of compression are provided. The gas is assumed to be sweet and to require no processing except dehydration. Interstage coolers with knockout drums are used to cool the gas. The heat generated from compression is utilized to heat the injection water.

High pressure gas is monitored and distributed through the manifold to the separate injection wells. The location of the gas injection module is next to the separation module and the flare. The last must be capable of flaring all injection gas in case of emergency.

The layout of the gas compression module for the 100 MBPD case developed for the steel jacket structure is shown in Figure 2-12 and for the 300 MBPD case in Figure 2-13 . There is one train of compression for 100 MBPD and three trains for 300 MBPD. The corresponding module for the 300 MBPD cases for the concrete monotower is shown in Figure

2.6.3 Water Injection and Ballast Water Treatment Module

As shown in Figure 2-15 for the concrete structure with base storage for crude, the water injection pumps are combined in one module with the ballast water treatment equipment. Water is to be injected into the formation in the proportion of one barrel of water for each barrel of crude oil produced. A savings can be realized on ballast water treatment requirements by injecting ballast water drawn off the storage tanks. When a tanker loads crude oil at an offshore berth in the vicinity of the platform, seawater is injected into the storage tanks as the oil is pumped out in order to maintain a small pressure difference across the tank walls. The continuing production of crude displaces the seawater ballast in the storage tanks into the ballast water treatment center, and all of this water is treated for injection into the reservoir. The incoming water is first heated at the interstage gas compression coolers, then passed through a corrugated plate interceptor, an induced air flotation unit, deareated, filtered and chemically treated. This module for the 100 MBPD case consists of three vertical water pumps and one de-aeration vessel measuring about 12 feet in diameter by 32 feet high. Low pressure gas is used to de-aerate the water in the proportion of about 0.5 MMSCFD of gas to 100 MBPD of water.

Additives and treatment tanks consist of anti-foam, biocide, coagulant and oxygen scavenger tanks.

Three main water injection pumps are indicated, and provisional space for a future one is allocated.

Monitoring and control of water quality and quantity injected are combined at the control panel. When crude is stored at a separate tanker loading structure, different options arise for handling of storage tank ballast water. Untreated oily water may be transported to the production platform by submarine pipeline for treatment and reservoir injection. Or, the oil can be separated at the loading structure with the emulsion delivered to the loading tanker and the clean water discharged into the sea. This option would require water to be drawn from the sea at the production platform for treatment and injection.

2.6.4 Wells Manifold and Wellhead Modules

The entire well piping manifold is conceived to be modularized in one or, at most, three units per drilling leg. An example of such a module is shown broadly generalized in Figure 2-16. The gas injection, oil production and water injection manifold sections may be separately placed on the platform.

One wellhead module is required for each drilling shaft - for a total of two in the case of the 4-legged structure. These modules are complete with wellhead templates (two levels), ladders, casing hangers and blind-flanged,inter-connecting piping to the manifold.

2.6.5 Power Generation Modules

Primary power will be generated by gas turbine units rated at about 25 MW for the 100 MBPD production case on the monotower structure. Two are shown in the power generation module (Figure 2-17). Normal electrical loads will require only one unit to be operating with the other on standby.

For the 4-legged steel structures, where two drilling rigs will be operating together, the system of two generators will be retained, but each unit will have a capacity of 35 MW (Figure 2-18) for the 100 MBPD case and 50 MW for the 300 MBPD case.

A waste heat recovery system will be used to recover heat from stack flue gases which may be then utilized for module space heating and process. The waste heat recovery system could be either an open-loop system if water is used, or a closed-loop system if glycol is used as the heat carrier. For the Bering Sea, a glycol system is preferred.

Emergency power during the production phase of the development will be provided by four diesel generator sets each rated at about 3 MW (Figures 2-19 and 2-20). These generators are the primary sources of power during the hookup and initial drilling phases of the platform operations.

2.6.6 Control Room Module

The control room module for the 100 MBPD production on the concrete monotower is shown in Figure 2-21 . This module also contains the telecommunications and the data processing area as a mezzanine above the telecommunications area.

Should the ALASCOM satellite system be used for pipeline control and platform to shore communications, the earth station electronics would also be located in the telecommunication room.

The control room module for a steel structure is shown in Figure 2-22.

2.6.7 Mainline Pumps

Mainline crude oil pumps are needed only in the option of a pipeline to shore. In the option of offshore tanker loading there are two alternatives:

- o concrete production structure with integral subsea storage or,
- o steel or concrete production structure with separate subsea storage in the tanker loading structure.

In the first alternative, oil transfer pumps will be located in the utility shaft. In the second, low-pressure, centrifugal pumps will be located on the production platform deck to transfer oil to the storage structure via short, seabed flowlines. Tanker loading pumps in this case will be located on the storage/loading structure.

The sizes and horsepower requirements of the mainline pumps vary according to the viscosity of the oil and the length and throughput of the pipeline to shore. The hydraulic gradient diagrams, Figures 2-23 through 2-26 give the pump discharge heads for the several examples treated. From these, the pump horsepower required can be determined directly.

2.6.8 Quarters and Utilities Modules

Accommodation and quarters module layouts are shown in Figure 2-27 and 2-28 . These modules are chosen to be identical among the various cases studied, whether the structure is a monotower or a multi-legged structure and whether the throughput is 100 MBPD or 300 MBPD. The case of larger throughput merely requires one additional quarters module to produce a structure of nine levels while the 100 MBPD case constitutes a structure of eight levels.

Additional quarters modules may be required depending on the details of the hookup plan as discussed in Section 2.10.2.1.

The first three levels are common to both throughputs. Five quarters modules are added for the 100 MBPD case and six quarters modules for the 300 MBPD case. Each quarters module accommodates 72 persons for a maximum of 360 persons in the former case and 432 in the latter. The need for so many beds is discussed in Section 2.10.2.1.

Full use of the available space is expected to occur only during the offshore hookup phase of the development and possibly into the early stages of drilling. Later, room occupancy can be reduced to two persons with the provision of separating each room into private quarters using curtain walls.

Some utilities are contained in the first level of the quarters structure (Figure 2-27). These are primarily intended to service the living quarters or to act as emergency units. The main platform utility module for a steel structure is laid out in Figure 2-22 . This contains -

- o The main boiler room
- o Firewater pumps
- o Foam tank
- o Sewage water treatment
- o Platform HVAC
- o Air compressors and utility air service
- o Waste compactor and disposal system.

Water desalinization units for the steel jacket alternative are located in a mezzanine of the power generation module (Figure 2-18).

2.6.9 Drilling Module(s)

There are 3 modules accompanying each drilling rig, the mud, storage and cement modules (Figures 2-29 and 2-30). One pipe rack module is shared by the two rigs and the space beneath is used as a drilling office and doghouse.

Each of the mud modules has at least two mud pumps. The capacity and rating of the mud pumps are selected by the drilling contractor.

Included with the mud tanks, whether active or reserve, are mud hoppers, agitators, cleaners, desander/desilters, degasser valves and centrifugal pumps. A shale shaker is assigned to each rig.

Mud additives are contained in the neighboring storage module for blending in the active mud tank.

One mud laboratory serves both rigs.

A work shop and a well logging unit are also contained in the storage module.

The cement module consists of pressurized cement pods of capacity ranging from 400-2800 cubic feet, a compressed air unit, cement mixer and a Halliburton unit comprising a steady flow separator and a recirculating mixer.

The derricks and draw works are fully-enclosed, skid-mounted units, one of which may be removed upon completion of drilling leaving the other for workover functions.

2.6.10 Other Topside Components

Other main components are the flare structure, the helideck pipe racks and three cranes, all shown in the main platform layouts of Figures 2-5 through 2-8 .

2.7 Payload and Area Summaries

Module layouts have been discussed in the preceding section. The major components of these modules and similar modularized topside components installed in the North Sea provide a basis for estimating the total topside weights. These are listed in Figure 2-31 for both 4-legged steel and concrete monotower support structures in each case of production throughput. Both the dry (lift) weight and the operating weight are identified for each module or topside component.

The maximum lift assumed for the derrick barge is 1600 tons. Modules or components which exceed this weight limit in the list are presumed to be made up of two or more separate lifts.

The last line of Figure 2-31 lists a derived quantity, the topside weight index, for each structure type and each rate of production. This quantity is the total topside weight including deck structure in metric tons divided by the intended crude oil throughput in units of thousands of barrels per day. This topside weight index plotted against throughput is shown in Figures 2-32 and 2-33 for the Bering Sea topside concepts as well as for a selection of topsides on North Sea platforms. A mean line is drawn through these points for both the dry and operating weights. Few data points are available for dry topside weight indices in the case of the steel support jacket; but, for the concrete structures, the dry topside weight index appears to be well defined.

As would be expected, the topside weight index decreases with increasing throughput. Also, the concrete structures show a higher weight index compared to the steel jackets, especially in the range of lower throughputs. It is of interest to note that the Statfjord developments A and B have not only operating weight indices well above the mean line, but even the dry weight index lies above the mean operating weight index of other North Sea concrete structures.

The four cases treated in this study for Bering Sea topside arrangements fall sensibly on the mean line for dry weights, but tend to lie above the mean for operating weights. This is consistent with the objective of providing more personnel accommodations, solid and liquid bulk storage, and weather protection than is required in the North Sea.

Floor space in each of the four topside alternatives is summarized in the table below. Mezzanine floor areas are shown separately.

FLOOR AREA ($Ft^2 x 10^3$)

	Multileg Steel Structure		Monotower Structure	
	100MBPD	300MBPD	100MBPD	300MBPD
Main Floor Area	127	137	142	146
Mezzanine Floor Area	6	10	10	11
Total Floor Area	133	147	152	157

2.8 Safety and Evacuation

In the North-Sea platform operations, free-fall or davit-lowered lifeboats are located along the edge of the platform's lower deck in sufficient quantity to evacuate all personnel on the platform, should the need arise. A safety ship should remain on station near the platform at all times to carry out pick-up and rescue operations if platform abandonment becomes necessary. Helicopters can be used to assist in the evacuation of platform personnel, but no dependence can be put on this means of transportation. Weather or emergency conditions on the platform can deny helicopter access. Moreover, the limited carrying capacity and lack of immediate availability of helicopters in times of emergency further downgrade their utility in platform rescue operations.

The safety of platform occupants is improved by having more than one platform structure interconnected by walkways to the production platform. This does not eliminate the need for personnel abandonment procedures, but it alleviates the demand for a timely response of a rescue action to an emergency situation.

North Sea procedures in safety and rescue operations mechanisms cannot be wholly adapted to the Bering Sea. The two are similar in the general inclemency of the weather, but the Bering Sea is different in one important regard - the presence of sea ice during the winter. As described elsewhere in this report, ice can intrude into all three lease basins. The ice can be particularly hazardous to the platform structure and to a forced evacuation of the platform when the edge of the ice pack is in the vicinity of the platform. Southerly winds and the waves they produce can put ice segments into motion with considerable force. Even under conditions of complete ice cover, the platform acts like an ice breaker, each leg leaving a trail of open water as the ice moves through.

Neither situation makes for an easy solution to the problem of emergency evacuation of the platform, but several considerations can be put forward.

A safety ship should be in attendance at all times, whether the platform is a part of a complex of structures or an isolated unit in the first phase of field development. During the open water season, this ship could be of the North Sea trawler hull design typified by the high bow for sea-keeping in short period, high amplitude waves. During the winter season, the safety ship should have an ice-breaking hull. The hull class would, of course, be determined by the ice conditions expected at the platform location.

The demands on the safety ship can be reduced by taking advantage of the ice "lee" created by the platform itself. That is, the ship can maneuver in the broken ice field behind the platform. This also reduces the chance of the ship being caught in the ice pack upstream of the platform and of then being carried into the platform itself. In this lee position, consideration should also be given to a constant tension winch attachment between the ship and the platform. The cable or chain connection would be left slack to hang below the ice cover and not freeze into the surface ice. The cable weight per unit length in water, the angle of the cable at the safety ship, the length of cable out and the cable tension provide all the information necessary for position control. An automatic response system could be developed on this concept for first order control on the main engines and steerage.

All actions of this system would be under the surveillance of the watch to manually over-ride the automatic station keeping system as required.

The ice breaker should be equipped with a helideck. One helicopter can be positioned on the deck while another is stowed. Under reasonable weather conditions, the helicopter can make short-hop transfers of personnel from the platform or the ice to the safety ship.

Successful abandonment of the platform requires that the men can get off under any weather conditions. Free fall systems are not always acceptable because of the possible presence of sea ice. However, passive, mechanical brake systems are available to limit the speed of descent of the rescue pod to sea level. This could be incorporated into the platform lifeboat systems with a provision for bypassing the brake in favor of free-fall during open-water conditions.

The final rescue action is the transfer of platform personnel to the safety ships from the platform lifeboats. The two operationally difficult situations are:

- o Open water during storm conditions.

- o Ice-covered water when either the solid ice cover is in transit with the platform legs breaking ice or partial ice cover prevails with open water separating the ice floes.

The first situation occurs in the North Sea. Current practices there for rescue at sea involve the use of Rigid Inflatable Boats (RIBs) stowed on the safety ship. These are composed of a solid, V-shaped hull fixed to an inflatable tube above the water line that serves as a combination gunwale and fender.* This rubber

* de Montaigu, E.H.; "RIBs Offer Stable Support." Offshore, March 1982, pp. 93-96.

collar not only gives good stability to the RIB in rough seas, but prevents damage or injury when the boat comes alongside a platform lifeboat or the safety vessel during personnel transfer.

During the open water season in the Bering Sea, several RIBs could be stowed aboard the safety vessel to be deployed as needed.

The situation of partial ice cover has not been experienced during evacuation of personnel from an offshore platform or, at least, has not been reported. Platform lifeboats of the type used in the North Sea would be unable to navigate through a broken ice field to the safety ship; and in a moving icefield, even though of complete cover, the lifeboats would be carried off in the direction of ice motion.

Instead of many small lifeboats, several large boats, with ice-strengthened hulls could be supported at the platform deck level. Even if unable to make way against the ice to the safety vessel, the boats would have sufficient life support capability and hull strength to maintain the occupants until a rescue could be effected.

For the most part, the platform rescue procedures in ice discussed above are speculative. Braking the lifeboat drops can clearly be done safely and reliably without further investigation into the mechanics of the operation. However, the slack wire control system suggested for the ice-breaking safety ship should be studied with a control system simulation and model tests under the kinds of environmental conditions expected at the platform location.

Consideration should also be given to the optimum size and numbers of ice-strengthened lifeboats under different abandonment scenarios giving regard to the large numbers of people proposed to be quartered on the Bering Sea platforms.

2.9 Resupply and Transportation Requirements

Bulk resupply of the offshore platform will be done with supply boats operating out of Herendeen Bay, Dutch Harbor or one of the other Unalaska Island sites described in Section 8.0. Most of these boats will have ice-breaking hulls and sufficient power to break ice of the thickness expected in their zone of operation.

Personnel transfer will be done by helicopter. For platforms in the North Aleutian Basin, an airport and helicopter base could be established on the north coast of the Alaska Peninsula; as, for example, the one shown in Figure 2-34 for Herendeen Bay.

If the offshore development is much further east towards Bristol Bay, it would be necessary to establish the helicopter base further east to minimize travel time. In this case a site like Wide Bay, one of those selected in Section 8.0, might be developed as a port of entry with on overland road connection to the airport on the north shore.

Developments in the St. George Basin would be better serviced by an airport/helicopter base located on Unalaska Island or Cold Bay.

If platforms are located at the northern section of the St. George lease area, it would be desirable to establish a secondary base on St. George Island by expanding the runway there for fixed-wing service and establishing helicopter fueling and maintenance facilities.

During the summer, St. George Island is frequently fogged in, and direct helicopter flights from the Alaska Peninsula or Unalaska to the St. George platform site may be required from time to time.

The use of picket ships with helidecks, radar and homing devices might be well-advised in the early stages of an offshore development in the Bering Sea. As operational familiarity builds up, these support vessels can be demobilized.

Offshore development of the Navarin Basin will require a helicopter base on St. Matthew Island. If it is not practical to construct a runway for large aircraft on St. Matthew, then a helicopter shuttle service from Univak Island, 145 miles to the east, can be substituted.

Sedco will be drilling COST wells in the Navarin Basin next year. Resupply will be by Boeing Vertol 234s flying out of Nome approximately 400 miles away. Although support procedures for exploratory operations are not necessarily practical for the support of permanent facilities, these operations should be watched for possible application to offshore developments in the Bering Sea.

2.10 Construction Planning

The platform topsides can be assembled on a steel support structure or on a concrete gravity base structure which may or may not have subsea storage of crude oil in its base. If storage cells are provided in the base of the concrete production platform, then a utility shaft must be built into the structure at the time of its fabrication. As discussed elsewhere in this report (Section 5.0), the utility shaft gives access to the storage cells and contains the pumps, piping and wiring needed to transfer oil and ballast water to and from the storage cells.

In the option of a steel support structure, the structure must be set and piled at the offshore site before the topside components can be placed on top. Especially in a harsh environment, it is essential to modularize these components and to make each module as large as available derrick lift capacity will permit. This is not only to minimize the offshore work required to assemble the topsides, but also to minimize the number of hook-up interfaces required to commission the platform.

When the support structure is a concrete gravity base, circumstances do not so rigorously define the procedures for topside assembly. The practice that has been generally followed has been to:

-- assemble the topsides and the deck structure nearshore, preferably in a shipyard.

-- fabricate the concrete base structure nearshore in protected waters.

-- tow both the completed or nearly completed deck structure and the base structure to a deepwater location nearshore where the base can be ballasted down for mating with the deck structure. At this third site which, though nearshore, is usually remote from industrial support facilities, further work is required to interface the topsides with the utility shaft components.

-- tow out the nearly completed platform to its offshore location and set it on the seafloor.

-- remove the seafastenings and prepare the platform for commissioning and start-up.

Considering that the work of topsides fabrication, assembly and hookup are dispersed through multiple locations over the whole construction schedule, there is no well-defined sequence of events for gravity base platforms as there is for steel jacket structures. Neither does a pattern emerge from the history of gravity-base platform constructions. Platforms with integrated topsides planned for completion nearshore have been towed out with much of the topsides not installed, either to meet schedule with the weather window or because the topside total weight had become too great for stability under tow.

For purposes of unambiguous construction planning and cost estimating of topsides on a concrete base, several assumptions must be made:

1. The complete deck structure and topside components will be assembled at a shipyard and 90% of the hookup work will be completed there.

2. The same topside modules intended for the steel jacket structures will be used for the topsides assembly at the shipyard. This is assumed for reasons of engineering economy in this study, but the question of modular versus integrated construction of topsides nearshore is one of degree and not of kind. At one end of the range is the fully modular topsides, as assumed here, where the number of module interfaces is a minimum of 40 to 50. There is also a maximum in module steel for structural strength in transportation and lift of the modules.

   The fully integrated deck lies at the other end of the range. Individual pieces of equipment are lifted aboard with no modular steel, but there results hundreds of interfaces to hookup.

3. The deck, 90% complete, will be barged to a remote, nearshore location where it will be mated with the concrete base structure. If there is no provision for oil storage in the structure, no topside hookup manhours will be predicated for this mating operation. If there are storage cells and a utility shaft in the structure, hookup manhours will be allotted for connecting the components of the utility shaft to the topsides at this remote location.

4. After the platform is towed out and set in place, the remaining 10% of the hookup manhours will be expended on the topsides. This is evidently not a plan of action but an allowance. Since no gravity structure has been fully completed nearshore due to delays in schedule and to the need for tow-out during the weather window, it would be unrealistic to assume a platform for the Bering Sea could improve on that record.

5. Consideration has been given by others to the question of how much work would be needed to commission a gravity platform if it were to be towed out in a state of fully completed hookup. From these considerations, an opinion has developed that a minimum of 500,000 manhours would be needed to remove sea fastenings, repair damage resulting from the tow and perform other works prior to platform commissioning and startup. It will be assumed in this study that for both cases of throughput, 500,000 additional manhours will be required offshore for preparation of the platform beyond the 10% allowed for hookup completion.

2.10.1 Topside Modules

In the broad sense, the design and construction of the topside modules can follow either of two approaches -

1. Complete, detailed design of all modules by a prime contractor who solicits bids from module fabricators based on the design drawings.

2. Conceptual design of the modules by a prime contractor who solicits bids for detailed design and fabrication from suppliers.

The second method is appropriate to a fast track schedule, since the module supplier, working from detailed specifications on module performance, interfacial connections, and size and weight requirements, is otherwise free to draw from previous designs and equipment selections to expedite the module fabrication. On the other hand, this approach requires a high level of supervision, coordination and inspection on the part of the prime contractor to maintain cost control and schedule among the module fabricators.

The topside modules would be built in the United States or Asia and then transported by barge to the staging area in the Bering Sea region or to the site selected for the gravity-platform topside assembly. Derrick-barge and support fleet operations for module placement at the site of the steel structure location are described in Section 4.0.

2.10.2 Topsides Hookup

2.10.2.1 Hookup Offshore

The steel jacket structure will be towed out, launched, set in place, piled and prepared for loading topside modules.

The usual practice in setting modules is to load all of them in one continuous operation in as short a time interval as weather conditions will permit. To this purpose, emphasis is placed in designing the modules for the maximum lift weight within the capacity of available floating derrick equipment, taken in this study to be 1600 tons.

The modules for small platforms can sometimes be accommodated on the deck of the derrick barge and transported to the site of installation in one load. The large number and total weight of the modules for both the 100 MBPD and 300 MBPD platform alternatives prohibit this convenience. The modules will be transported from the staging area at Unalaska Island or Herendeen Bay to the platform site on either their ocean transport barges or on other vessels to which they will have been off-loaded earlier.

It is assumed that most of the modules would be loaded onto one end of the platform and skidded into place. This saves time in relocating the derrick barge to different positions around the platform between module transfers.

The typical procedure in North Sea construction is to anchor a floatel next to the platform for personnel accommodation during the entire period of hookup. This is not considered practical for Bering Sea operations because of the problems associated with maintaining a floating structure near a fixed platform in the presence of sea ice. Instead, sufficient living quarters are provided in the platform conceptual designs for all the hookup personnel. Even more bed spaces can be made available by stacking half-modules (36 beds each) if this measure is warranted. However, there is an interim period between completion of the module placement and the start-up of the life support facilities on the platform when workers may have to be quartered elsewhere for lack of accommodation on the platform. Alternative measures during this period include -

- Quartering the initial utilities hookup crew on the safety ship and transferring shift changes to the platform by helicopter.

- Retaining the derrick vessel for use as a temporary floatel until life-support systems are operational on the platform.

- Designing some of the platform accommodation modules to be completely self-contained in power and life-support facilities. These could later be disassembled and removed from the platform once the permanent systems are operational.

The first mode of operation has been used before, and, in that instance, it was found to be unsatisfactory. Transfer delays due to weather and other schedule interruptions significantly slowed the work.

The third approach implies a limited working staff until full platform quarters and utilities are operational.

Temporary use of the derrick barge as a floatel is plausible only if that barge is a semi-submersible. Flat-bottomed barges and ship-shaped hulls are too active in a seaway to be moored near a fixed platform. In the particular case of a platform in the Navarin Basin with a pipeline to shore, construction planning includes two semi-submersible laybarges for the pipelay. One of these could be converted to a temporary floatel.

The first alternative was used for cost estimation. No floatel is costed.

2.10.2.2 Hookup Nearshore

Much of the initial hookup at sea scheduling discussed in the last section is simplified if most of the hookup is done nearshore.

In the case of tow-out with topsides completed or mostly completed, the quarters and utilities can be made operational soon after the platform is landed on site. This eliminates the need for temporary quartering during the initial phases of hookup, and the full complement of hookup staff, housed aboard the platform, can be given to readying the system for drilling.

2.11 Engineering Considerations

The conceptual topside designs developed in this study are postulated on particular properties of the crude oil. Significant deviations in the gas oil ratio, H_2S and CO_2 content and the water cut from the quantities assumed could result in important differences in the topside production facilities required.

It was implicitly assumed that wellhead pressure will be sufficient to produce the oil. Further facility cost considerations will be required if gas lift or downhole pumping is needed for crude production.

2.12 Summary and Conclusions

It may be generally concluded that production platform topsides in the Bering Sea are essentially the same as those in the North Sea with certain exceptions, all relating to the presence of sea ice in the Bering Sea. For the Bering Sea these exceptions are:

-- More quarters for personnel are required.

-- More storage capacity is needed for both bulk and liquids.

-- The maximum number of well slots is limited by the requirement that the conductors be protected from sea ice.

-- Platform evacuation procedures and equipment require development to ensure personnel safety during an emergency in the presence of sea ice.

-- Air and sea transportation is more expensive and more difficult due to ice conditions, fog and potentially long travel distances in addition to the consistently harsh climatic conditions.

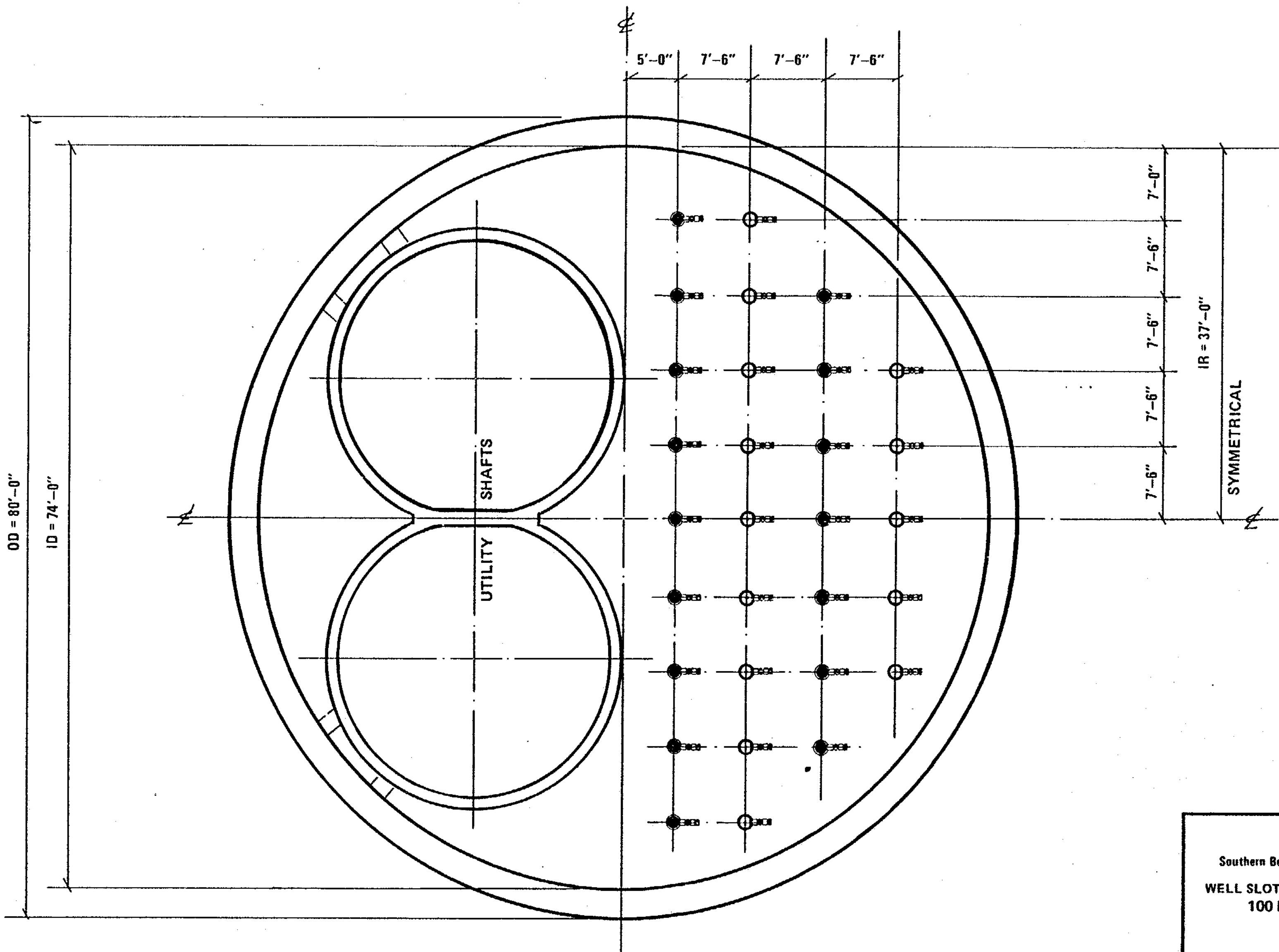
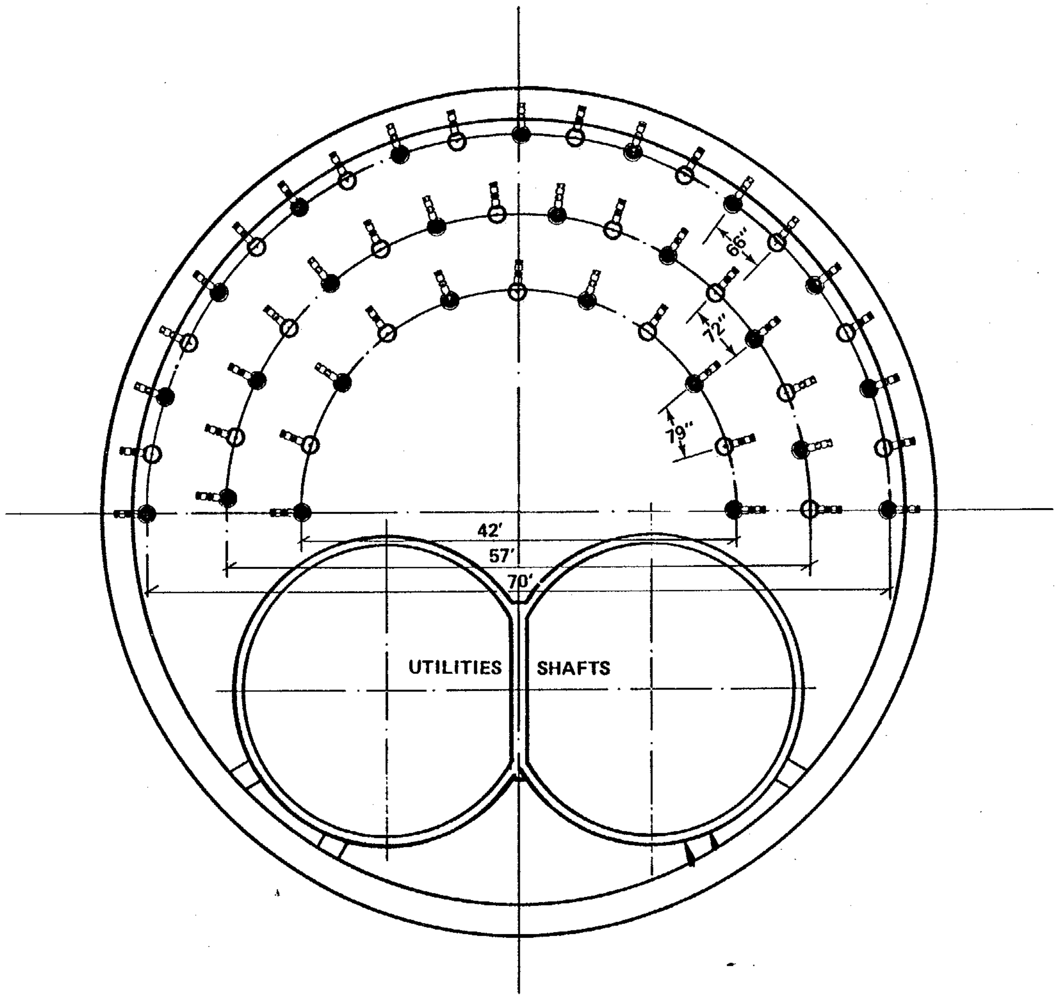


Figure 2-1
Southern Bering Sea Production Systems Study
WELL SLOTS, CONCRETE MONOTOWER
100 MBPD PRODUCTION

Job 15154 June 1982



MONOPOD LEG
OUTSIDE DIAMETER = 80'
INSIDE DIAMETER = 74'
NO. OF WELLS = 48

Figure 2-2

Southern Bering Sea Production Systems Study

WELL SLOTS, CONCRETE MONOTOWER
300 MBPD PRODUCTION

Job 15154 June 1982

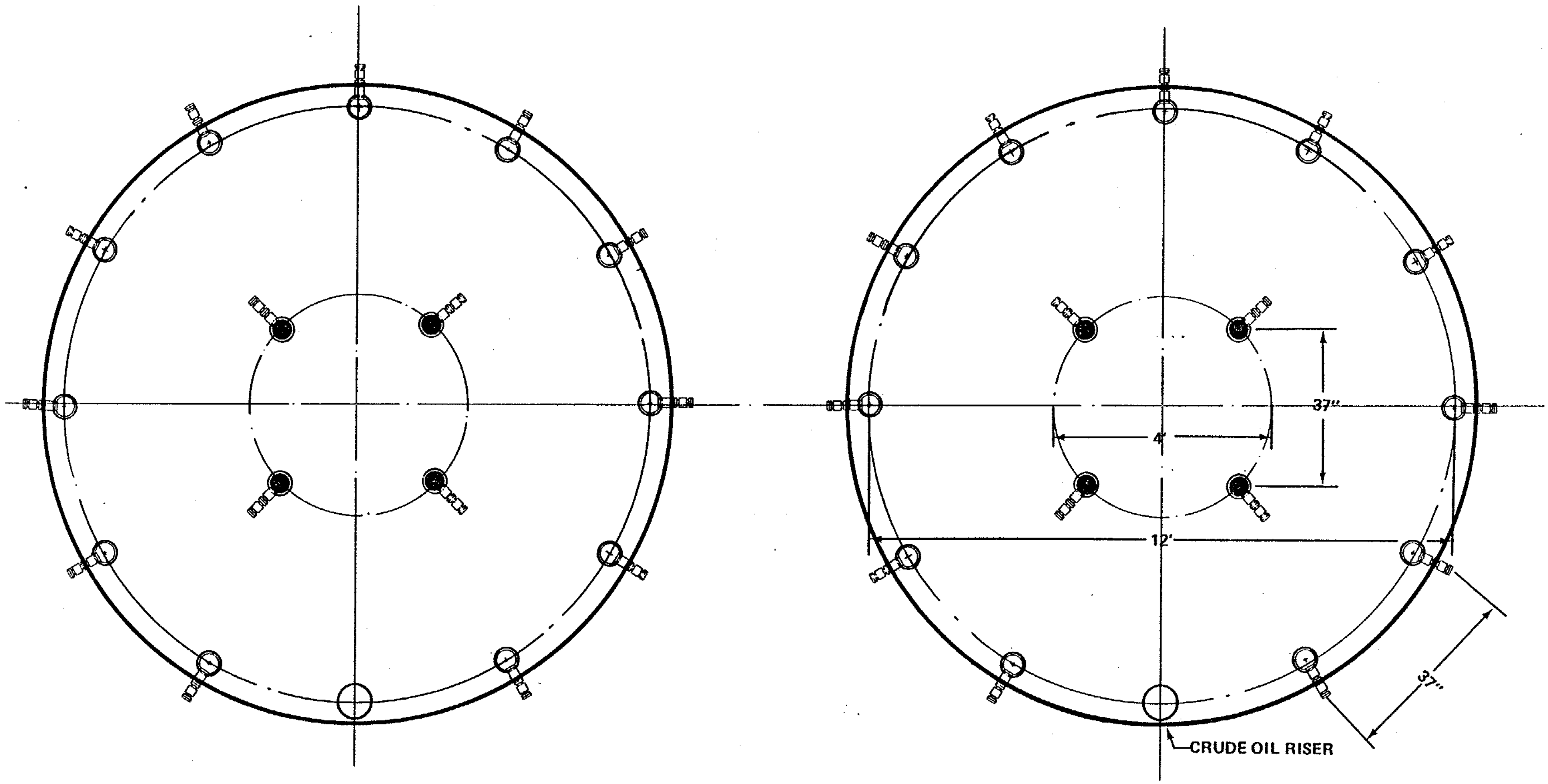


Figure 2–3

Southern Bering Sea Production Systems Study

WELL SLOTS, STEEL STRUCTURE
100 MBPD PRODUCTION

Job 15154 June 1982

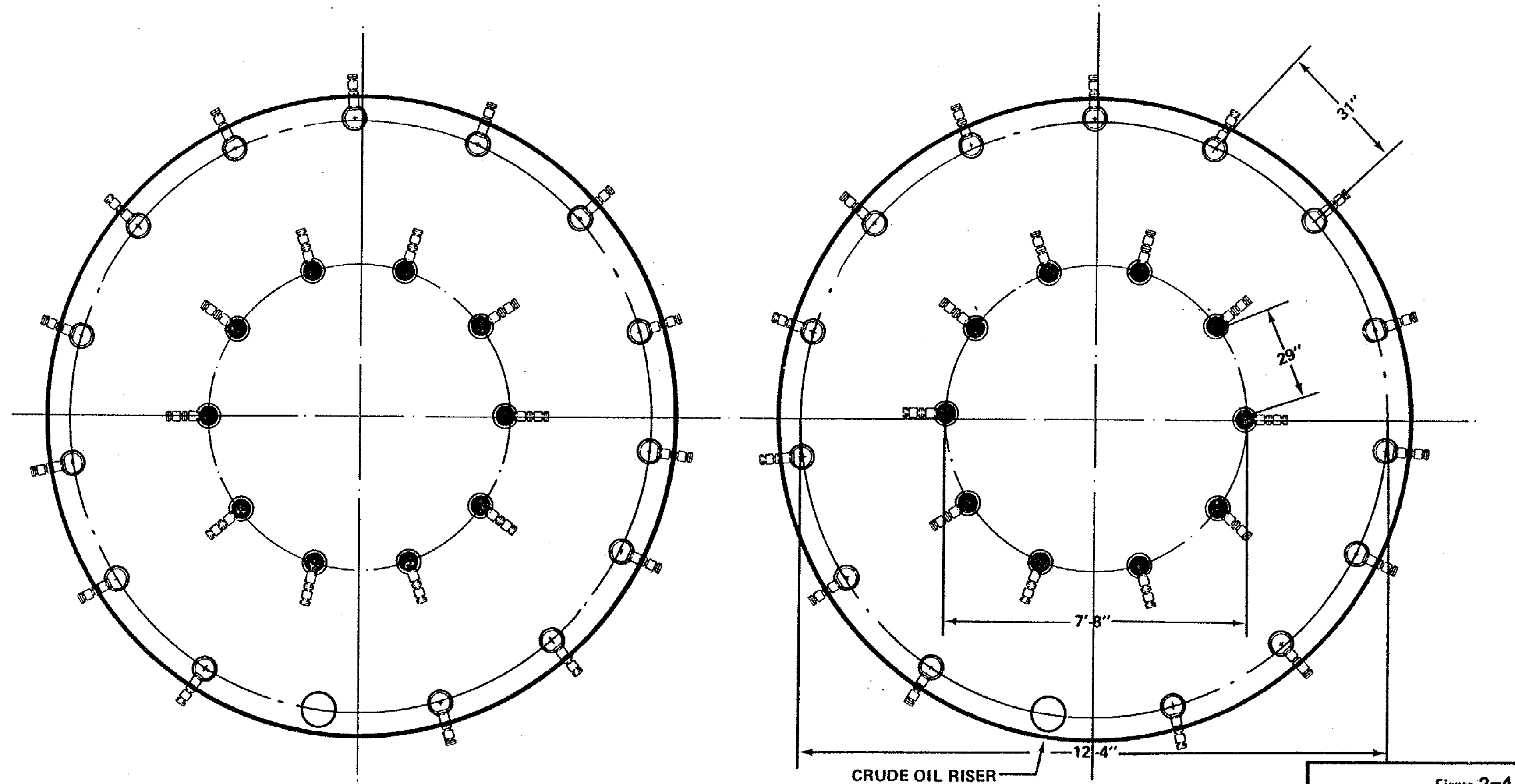


Figure 2–4

Southern Bering Sea Production Systems Study

WELL SLOTS, STEEL STRUCTURE
300 MBPD PRODUCTION

Job 15154

June 1982

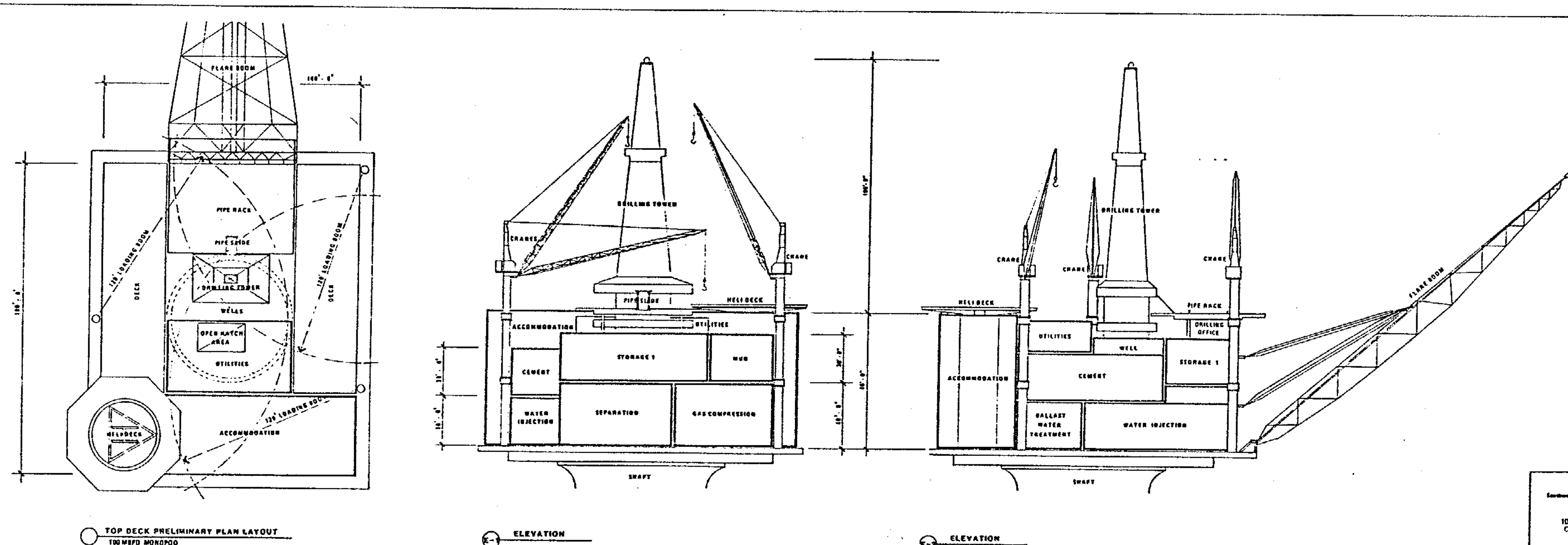


Figure 2-5

TOPSIDE LAYOUT
100 MBPD PRODUCTION—
CONCRETE MONOTOWER

Job 15154 June 1982

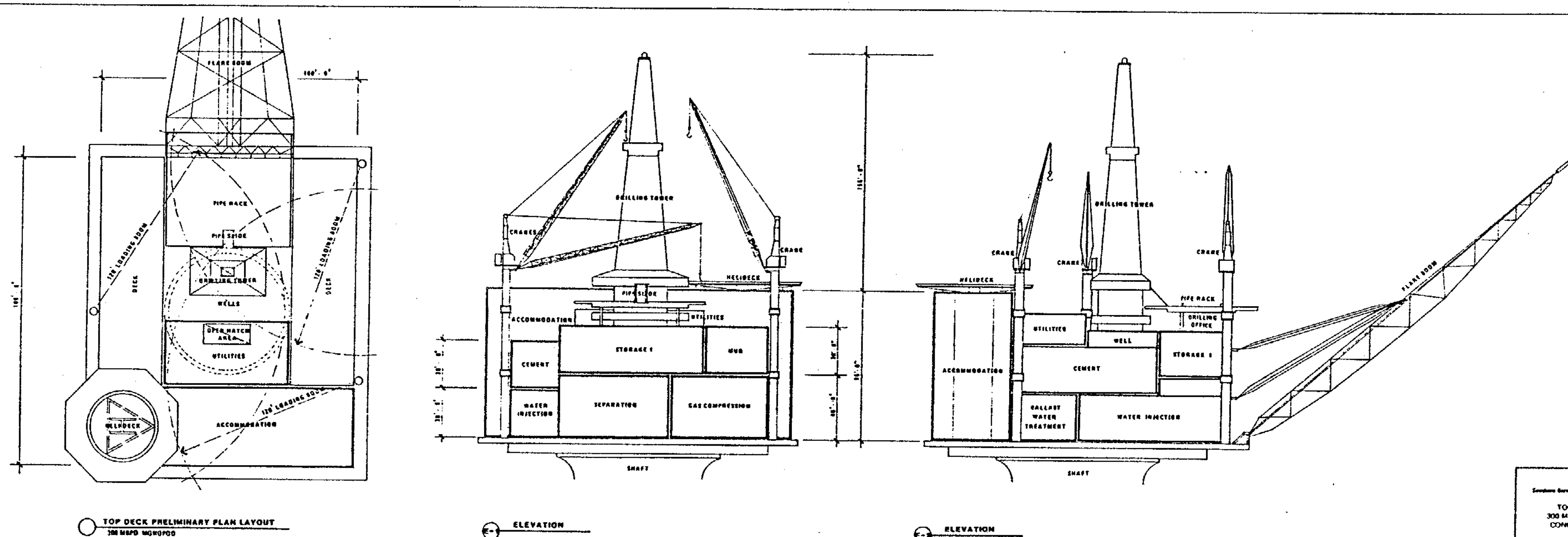


Figure 2-6

TOPSIDE LAYOUT
300 MBPD PRODUCTION—
CONCRETE MONOTOWER

Job 15154 June 1982

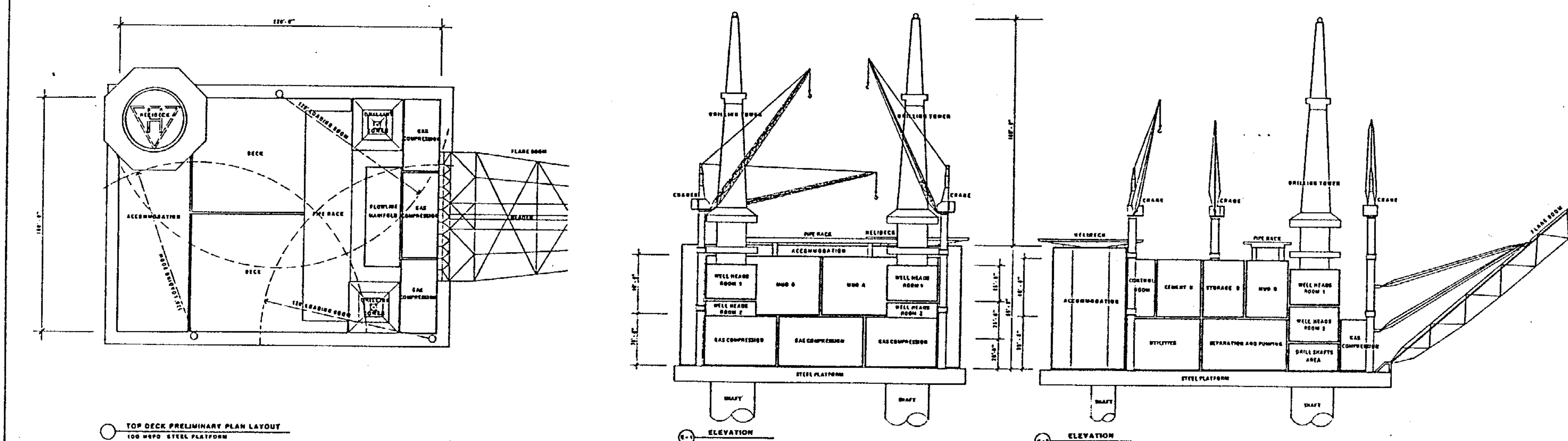
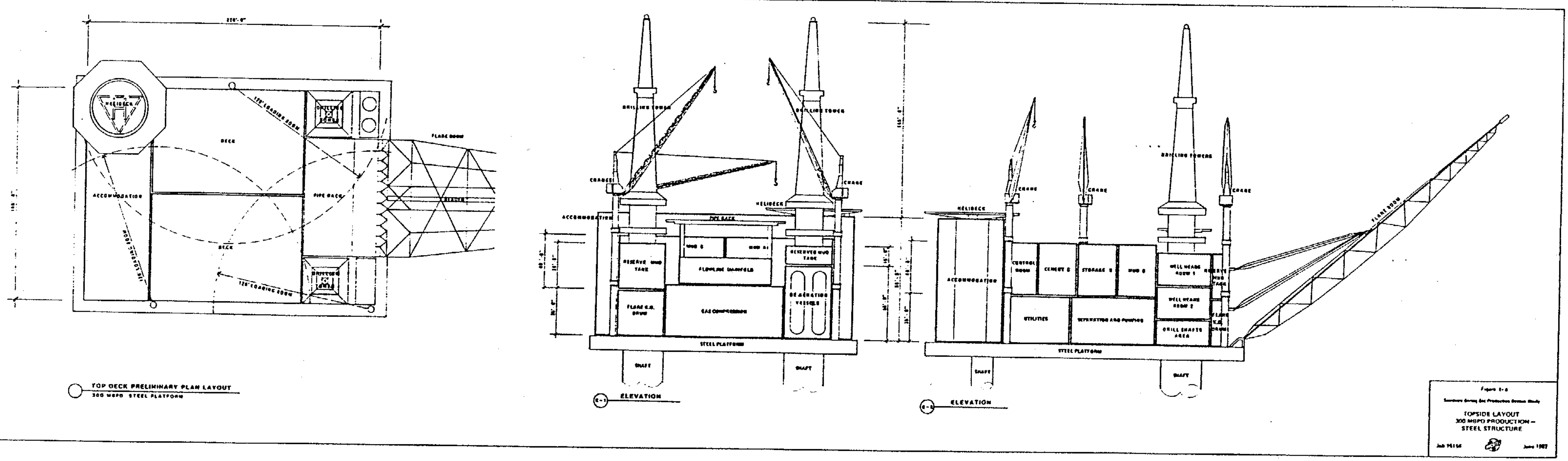


Figure 2-7

TOPSIDE LAYOUT
100 MBPD PRODUCTION–
STEEL STRUCTURE

Job 15154 June 1982



TOP DECK PRELIMINARY PLAN LAYOUT
300 MBPD STEEL PLATFORM
DECK
ACCOMMODATION
PIPE RACK
FLARE BOOM
ELEVATION
DRILLING TOWER
CRANE
HELIDECK
PIPE RACK
RESERVE MUD TANK
FLOWLINE MANIFOLD
GAS COMPRESSION
FLARE K.O. DRUM
DEAERATION VESSELS
STEEL PLATFORM
SHAFT
ELEVATION
ACCOMMODATION
CONTROL ROOM
UTILITIES
SEPARATION AND PUMPING
WELL HEADS ROOM 1
WELL HEADS ROOM 2
DRILL SHAFTS AREA
DRILLING TOWERS
FLARE BOOM
TOPSIDE LAYOUT
300 MBPD PRODUCTION —
STEEL STRUCTURE

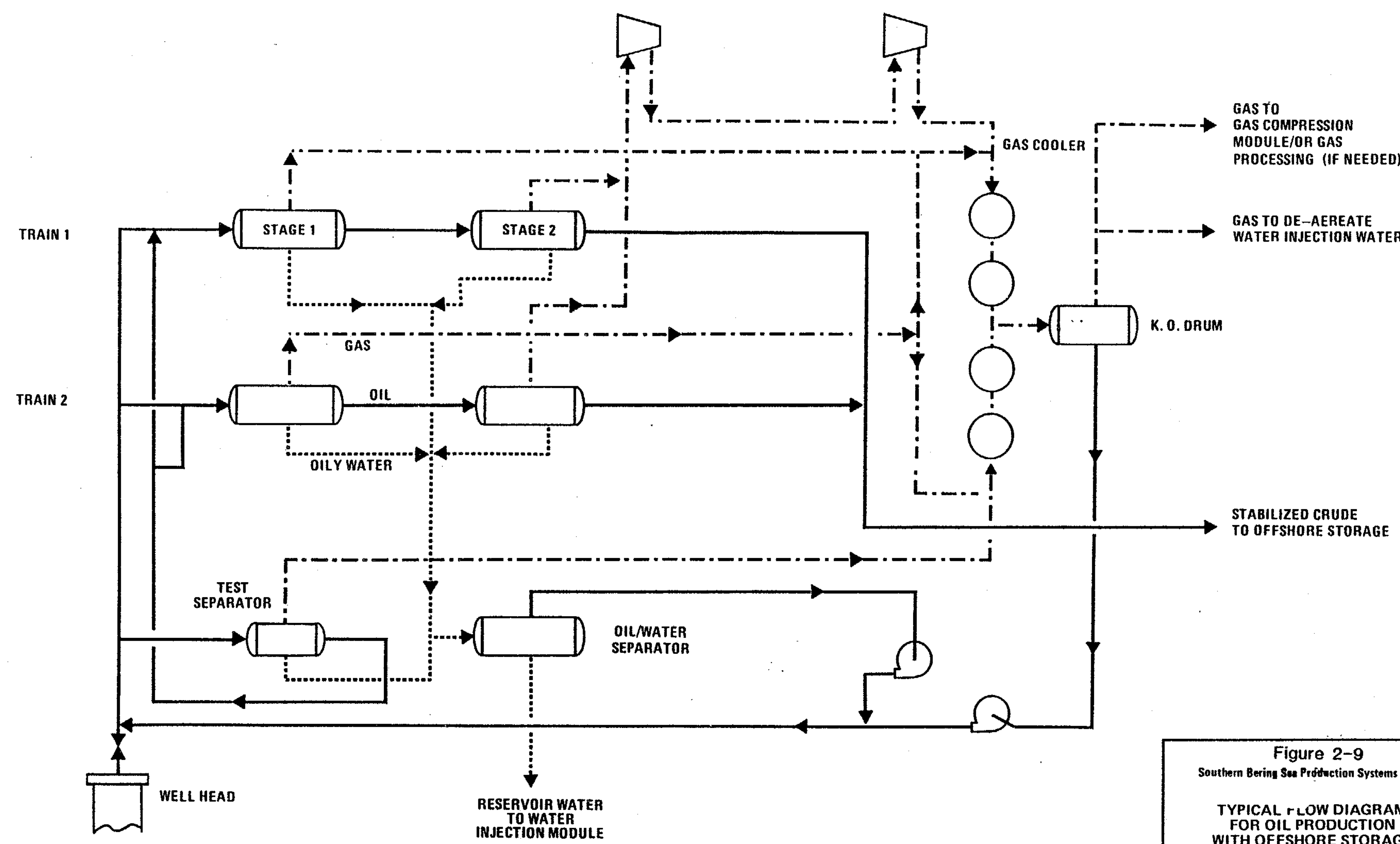


Figure 2-9
Southern Bering Sea Production Systems Study

TYPICAL FLOW DIAGRAM
FOR OIL PRODUCTION
WITH OFFSHORE STORAGE
300 MBPD

Job 15154 June 1982

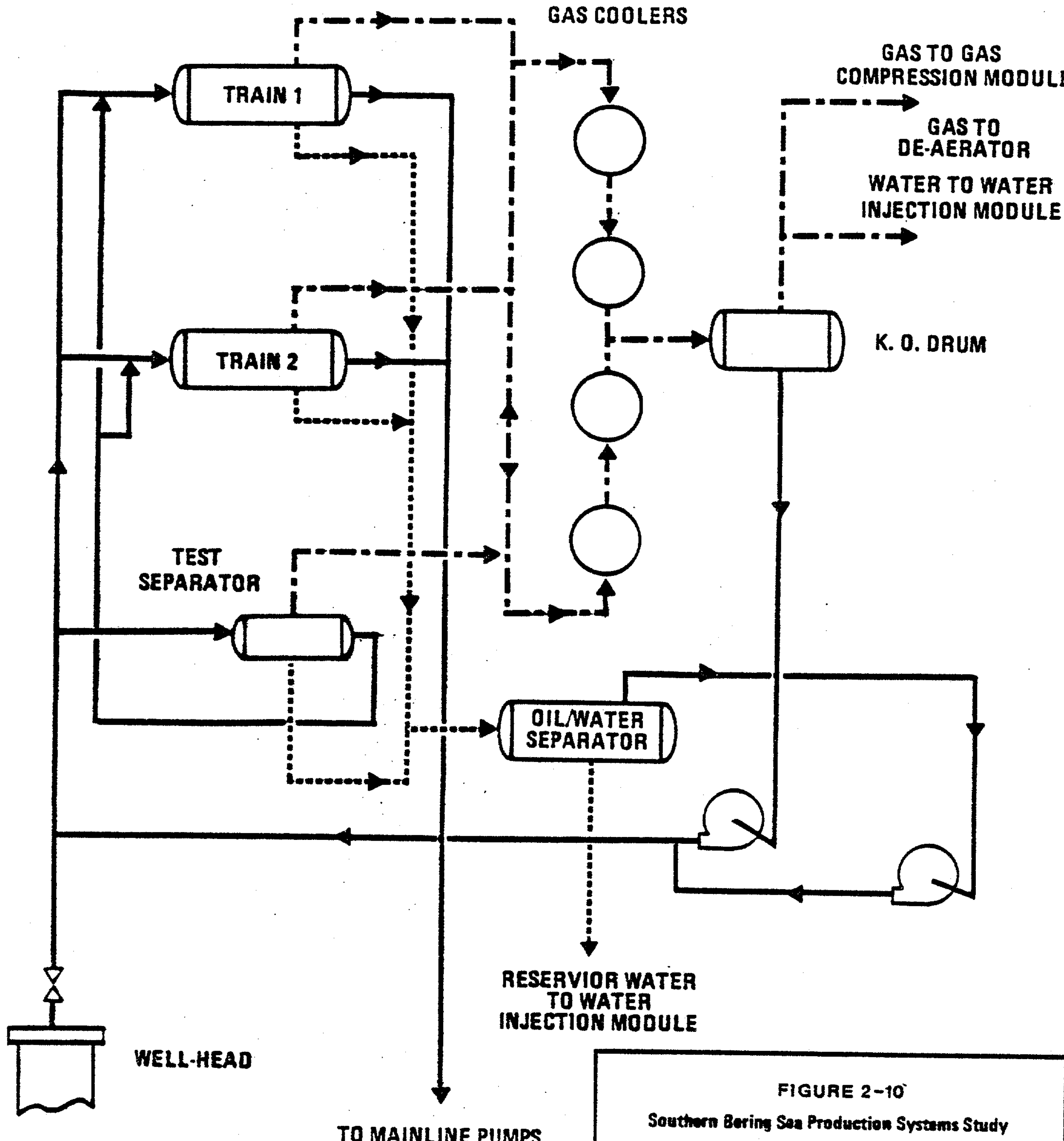
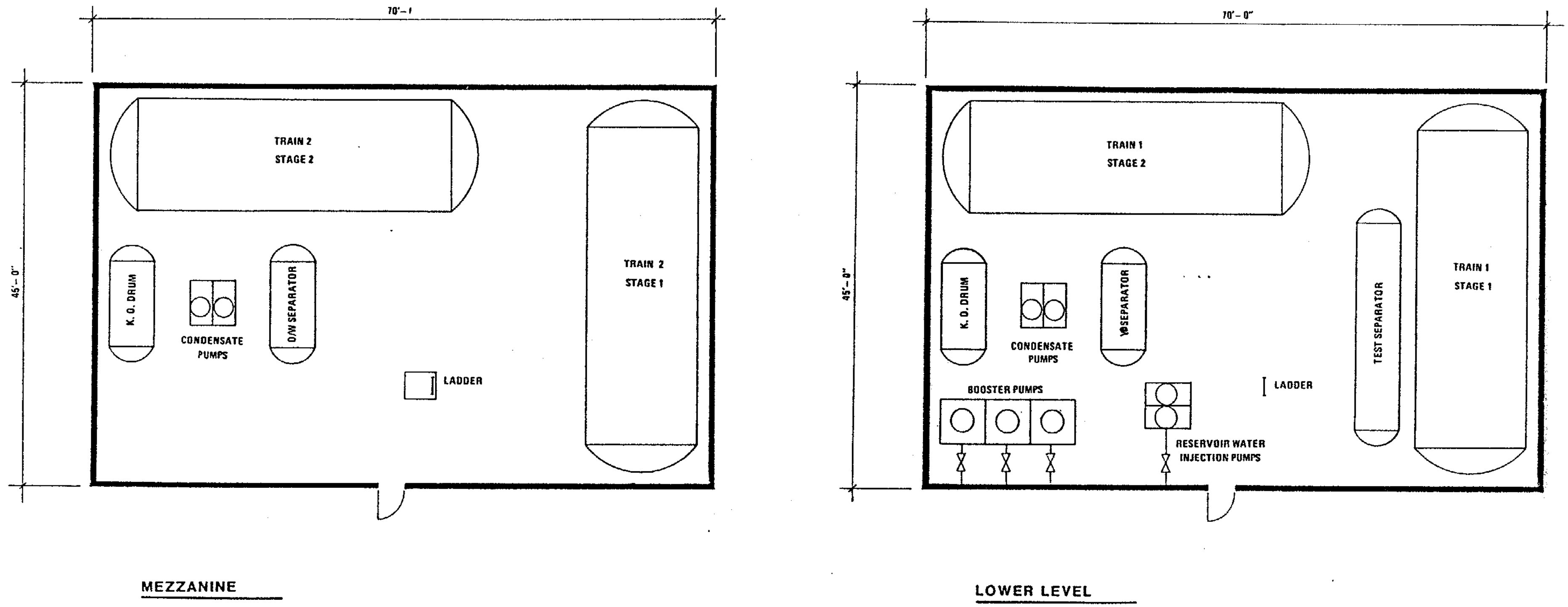


FIGURE 2-10

Southern Bering Sea Production Systems Study

TYPICAL FLOW DIAGRAM
FOR OIL PRODUCTION
WITH ONSHORE STORAGE
300 MBPD

Job 15154 BECHTEL June 1982



SEPARATION MODULE
100 AND 300 MBPD MONOTOWER

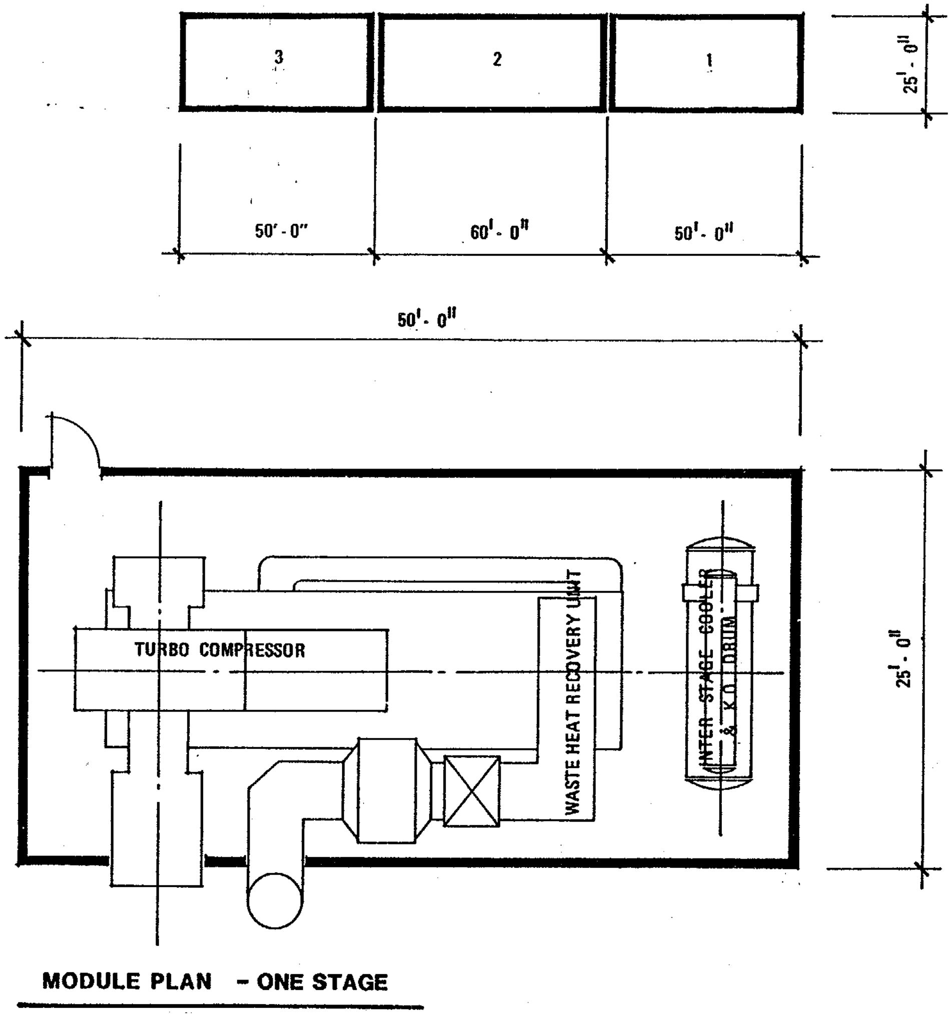
NOTE: ONLY LOWER LEVEL EQUIPMENT IS REQUIRED FOR 100 MBPD.
MEZZANINE LEVEL REQUIRED FOR SECOND.
TRAIN IN 300 MBPD CASE.

Figure 2-11

Southern Bering Sea Production Systems Study

SEPARATION MODULES FOR 100 & 300 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982



NOTE: EACH MODULE CONSISTS OF:

1. GAS TURBINE
2. COMPRESSOR
3. GAS COOLER AND SCRUBBER

GAS COMPRESSION MODULE

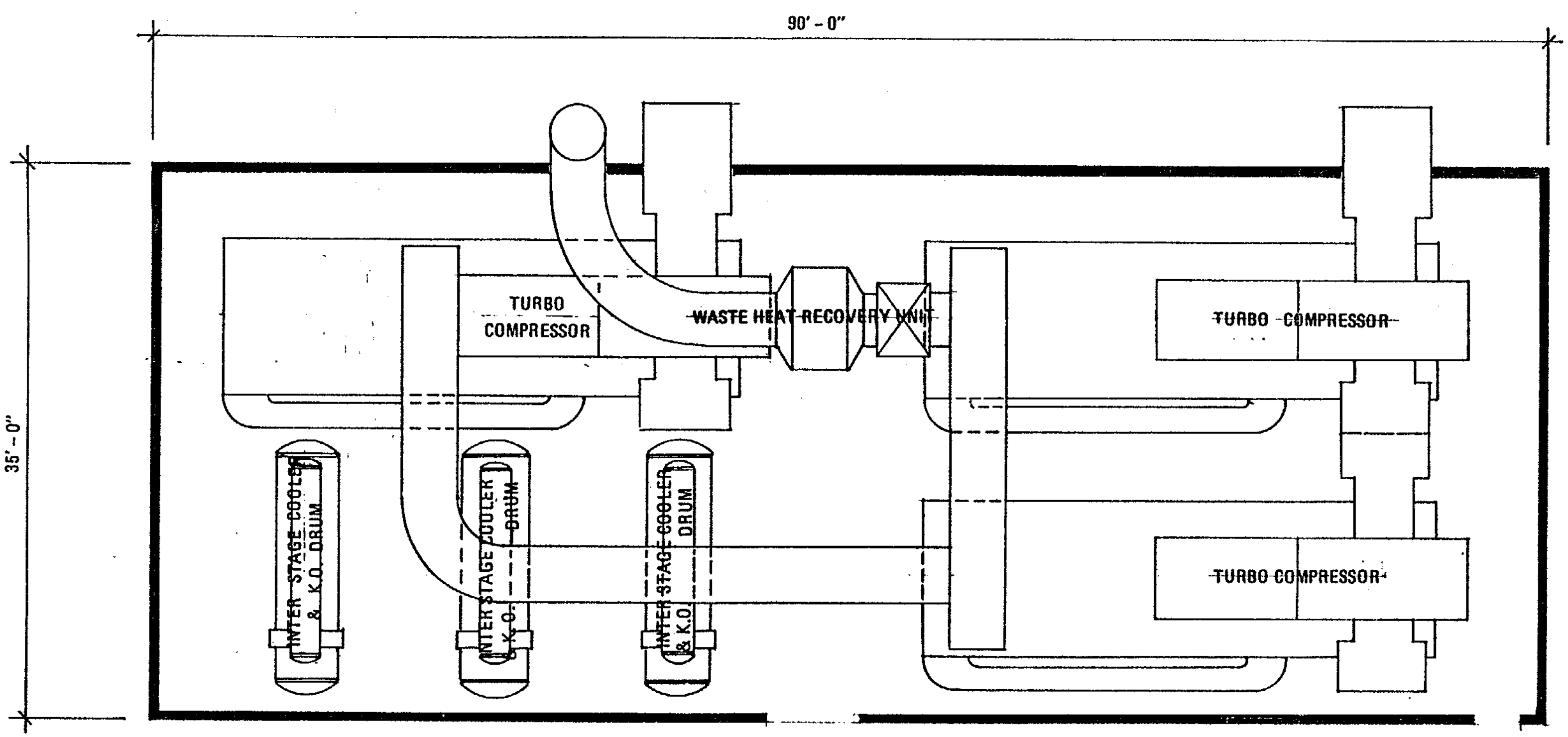
100 MBPD PRODUCTION STEEL STRUCTURE

Figure 2-12

Southern Bering Sea Production-Systems Study

GAS COMPRESSION MODULE FOR 100 MBPD PRODUCTION-STEEL STRUCTURE

Job 15154 June 1982



GAS COMPRESSION MODULE

300 MBPD STEEL STRUCTURE

ONE OF THREE PARALLEL TRAINS

Figure 2-13

Southern Bering Sea Production Systems Study

GAS COMPRESSION MODULE
FOR 300 MBPD
PRODUCTION-STEEL STRUCTURE

Job 15154 June 1982

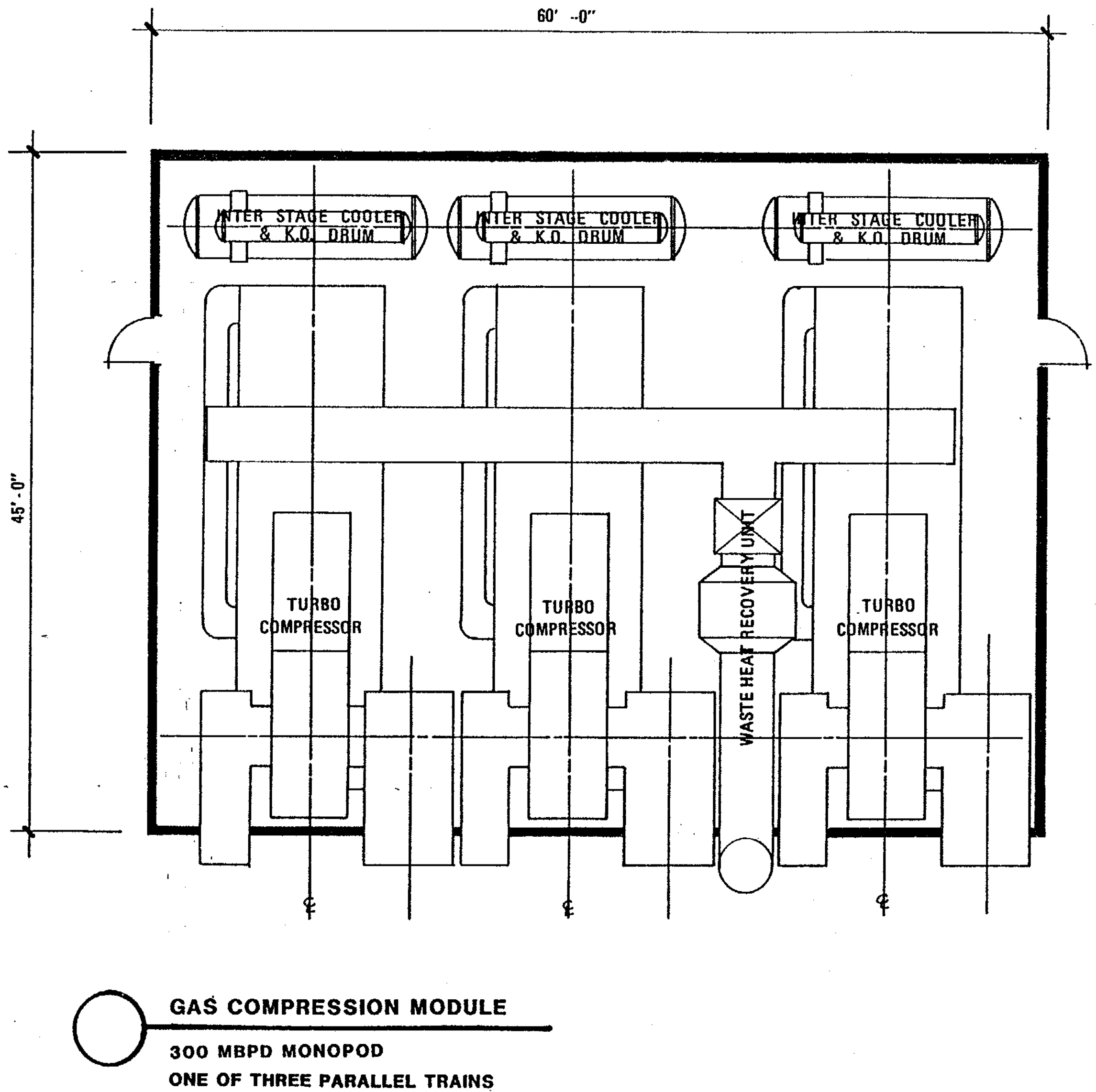
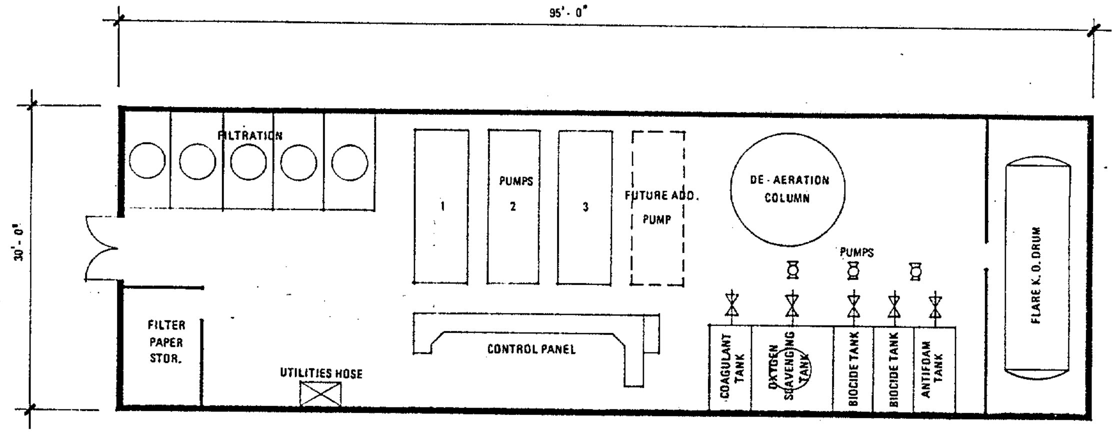


Figure 2-14

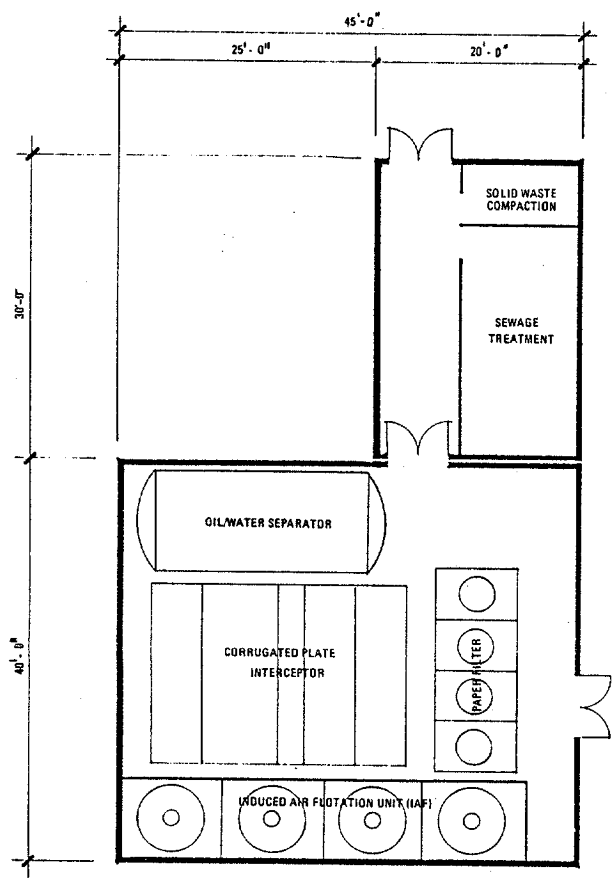
Southern Bering Sea Production Systems Study

GAS COMPRESSION MODULE
FOR 300 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982



WATER INJECTION MODULE
100 MBPD MONOTOWER



BALLAST WATER TREATMENT MODULE
ONE LEVEL FOR 100MBPD & ADDITIONAL MEZZANINE FOR 300 MBPD

Figure 2-15

Southern Bering Sea Production Systems Study

WATER INJECTION AND BALLAST WATER TREATMENT MODULES FOR 100 & 300 MBPD PRODUCTION-CONCRETE MONOTOWER

Job 15154 — June 1982

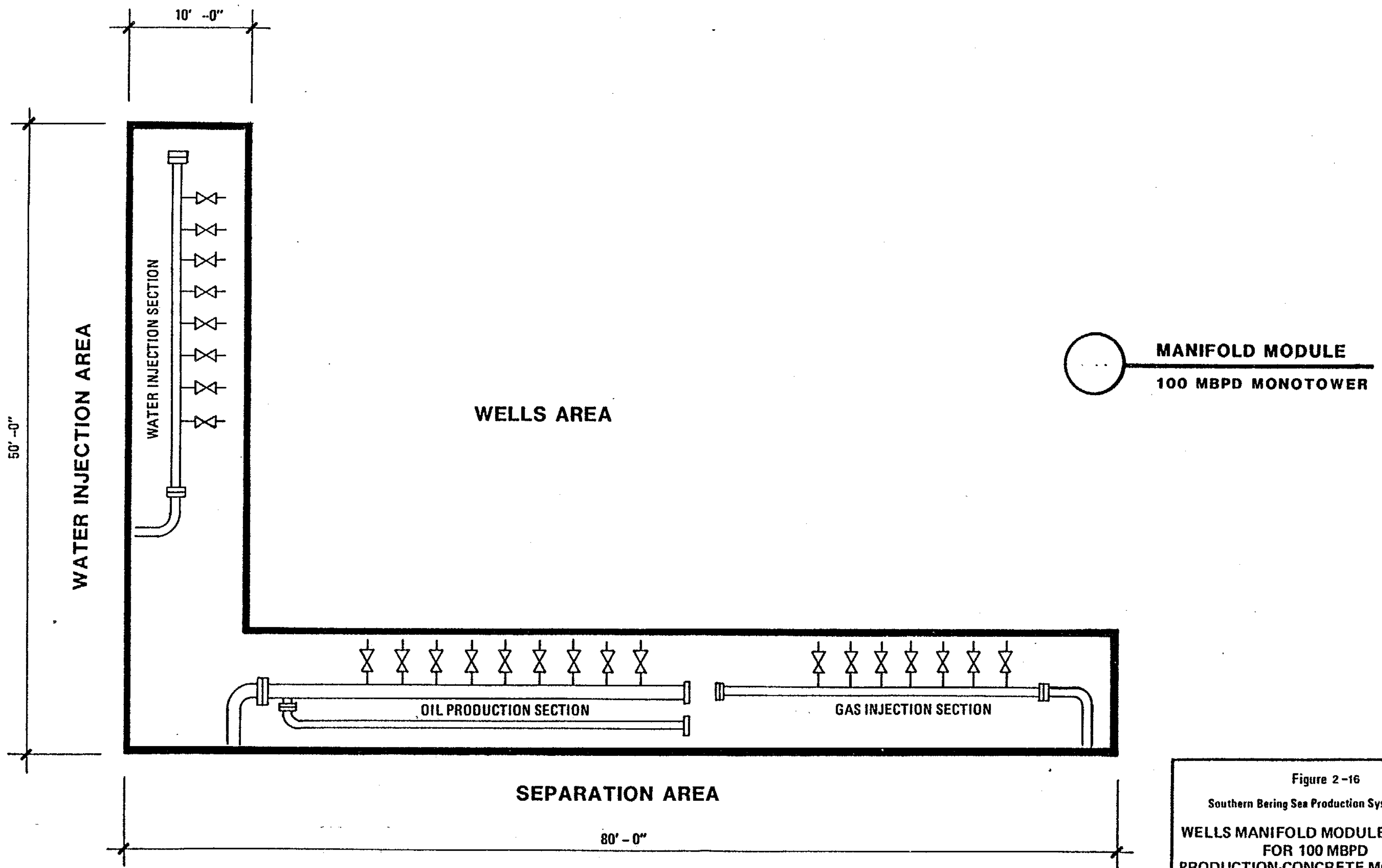


Figure 2-16

Southern Bering Sea Production Systems Study

WELLS MANIFOLD MODULE (TYPICAL)
FOR 100 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982

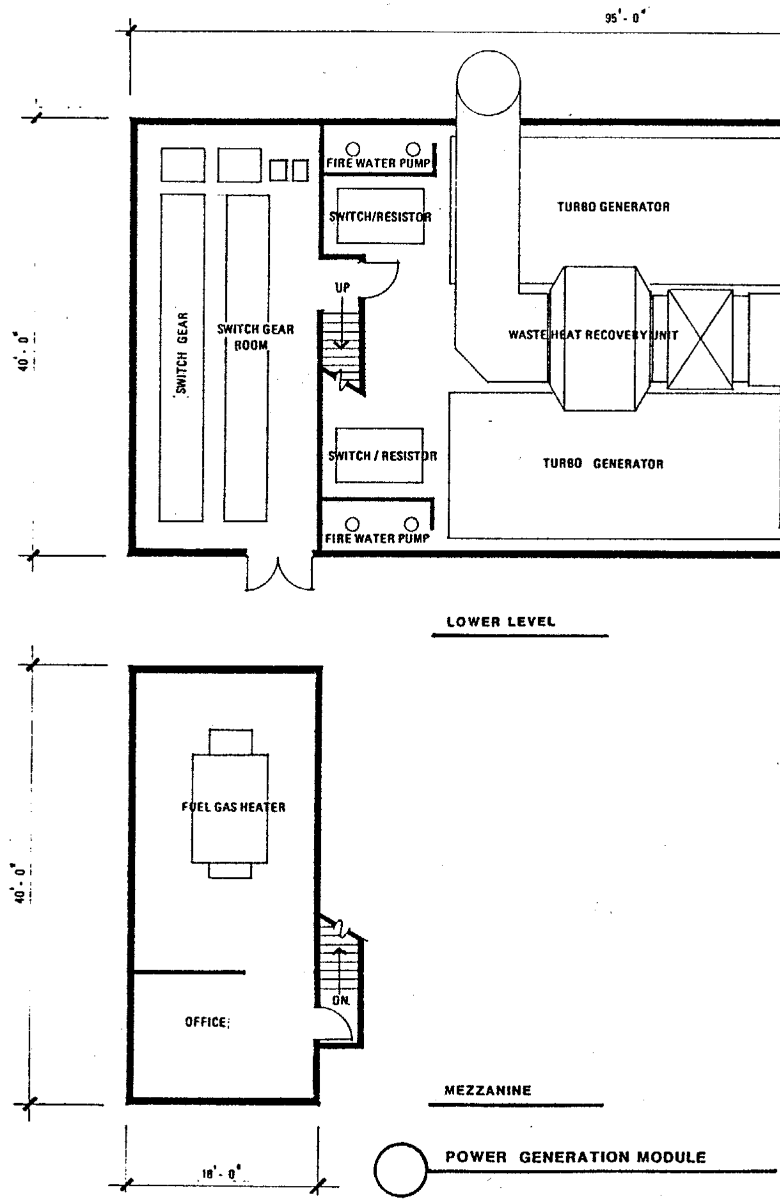


Figure 2-17

Southern Bering Sea Production Systems Study

POWER GENERATION MODULE
FOR 100 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982

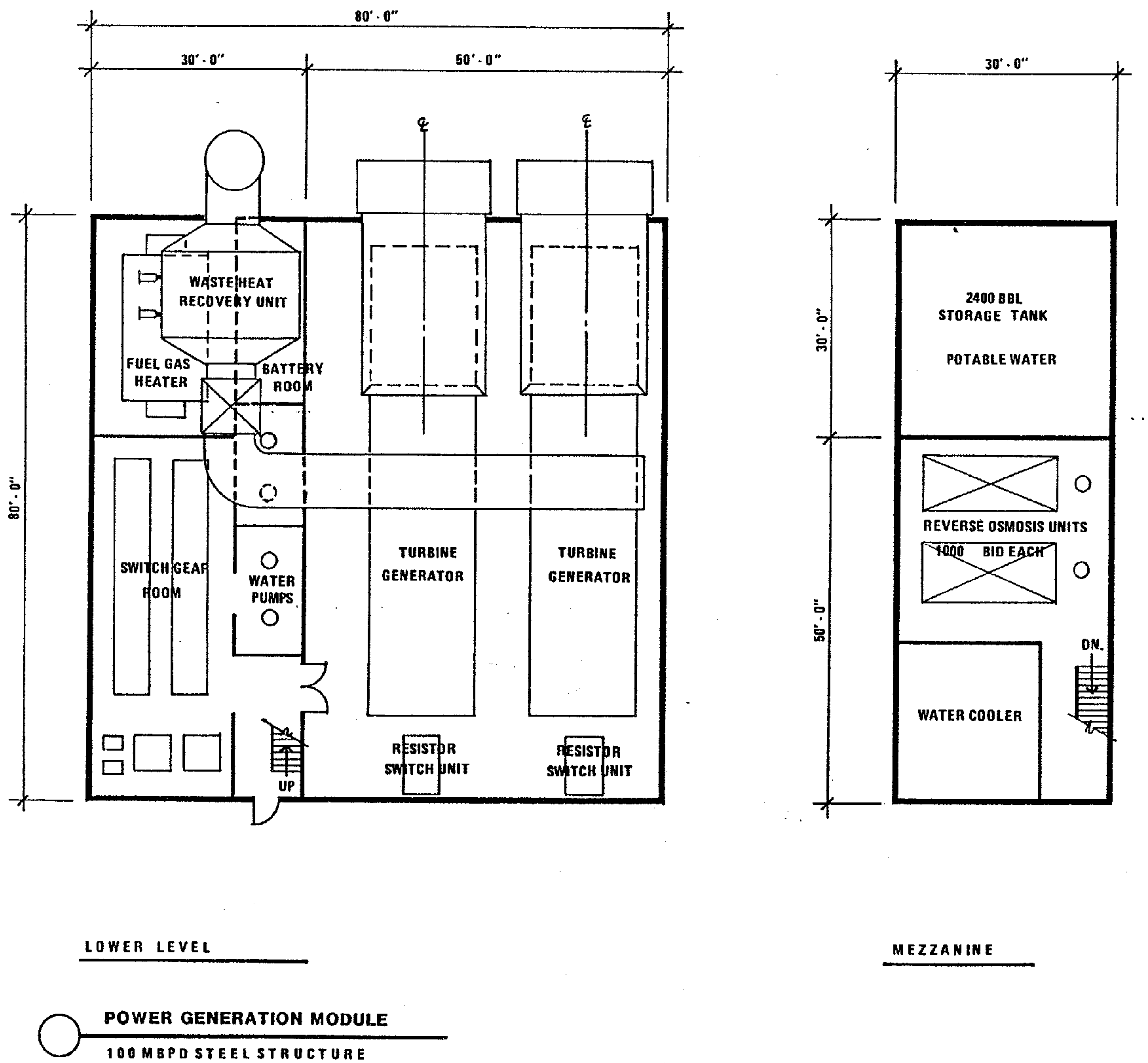


Figure 2-18

Southern Bering Sea Production Systems Study

POWER GENERATION MODULES FOR 100 MBPD PRODUCTION-STEEL STRUCTURE

Job 15154 June 1982

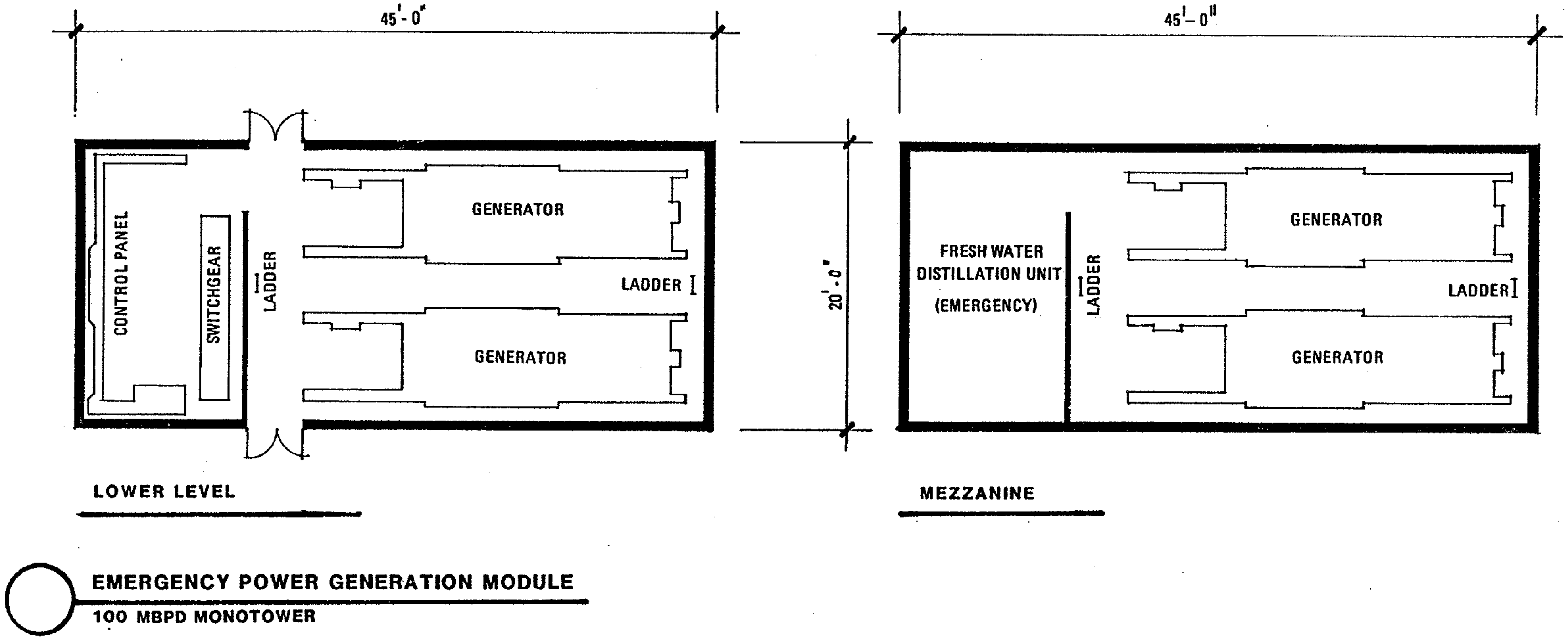
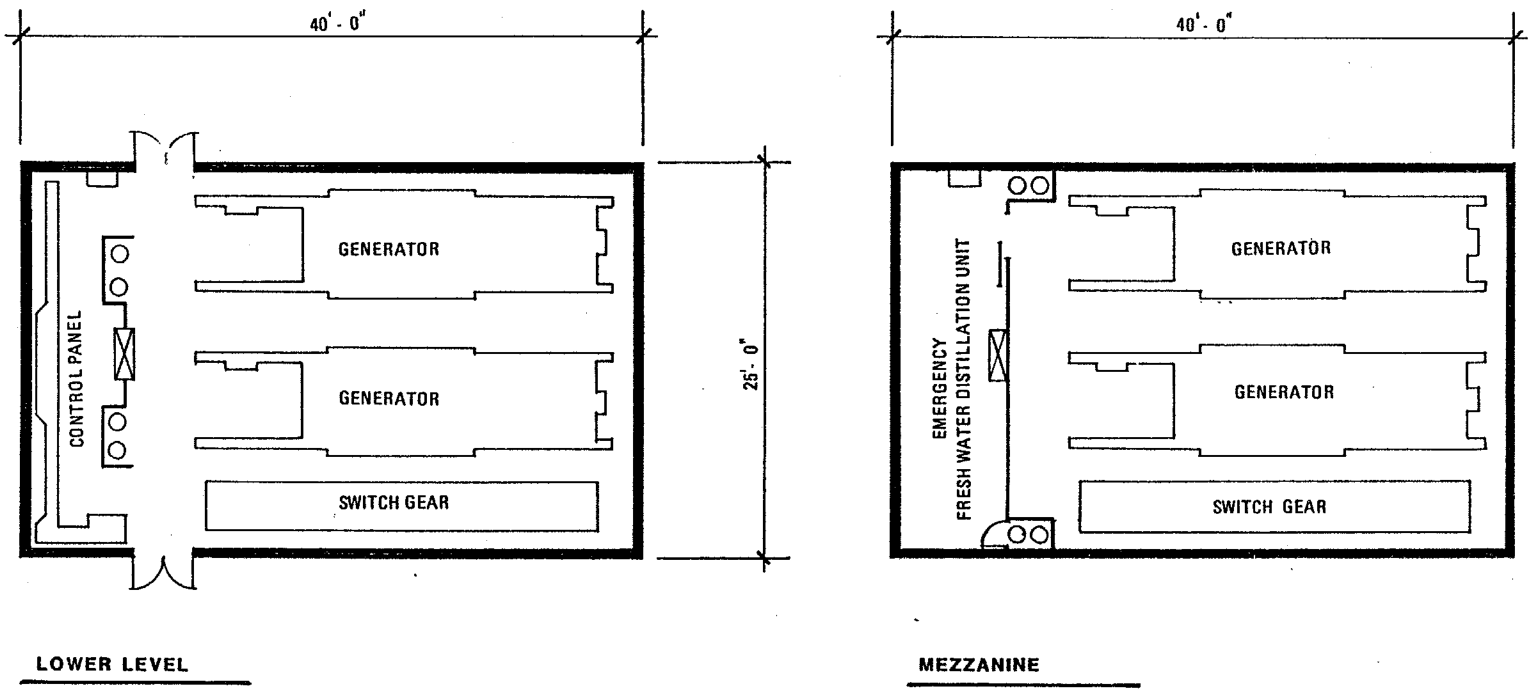


Figure 2-19

Southern Bering Sea Production Systems Study

EMERGENCY POWER GENERATION
FOR 100 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982

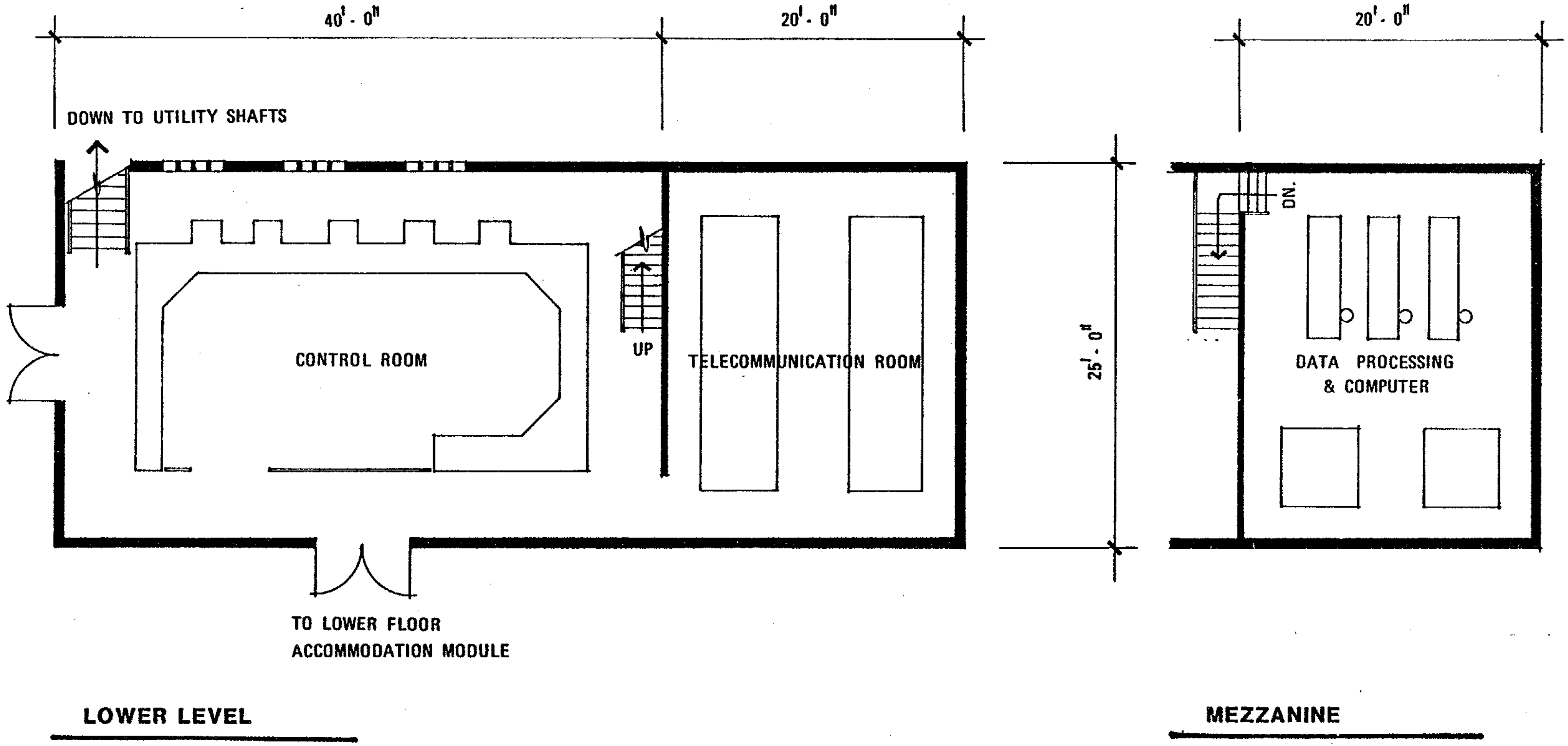


EMERGENCY POWER GENERATION MODULE
300 MBPD MONOTOWER

Figure 2-20
Southern Bering Sea Production Systems Study

EMERGENCY POWER GENERATION MODULE FOR 300 MBPD MONOTOWER

Job 15154 June 1982



CONTROL ROOM MODULE

100 MBPD MONOTOWER

Figure 2 - 21

Southern Bering Sea Production Systems Study

CONTROL ROOM MODULE
FOR 100 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982

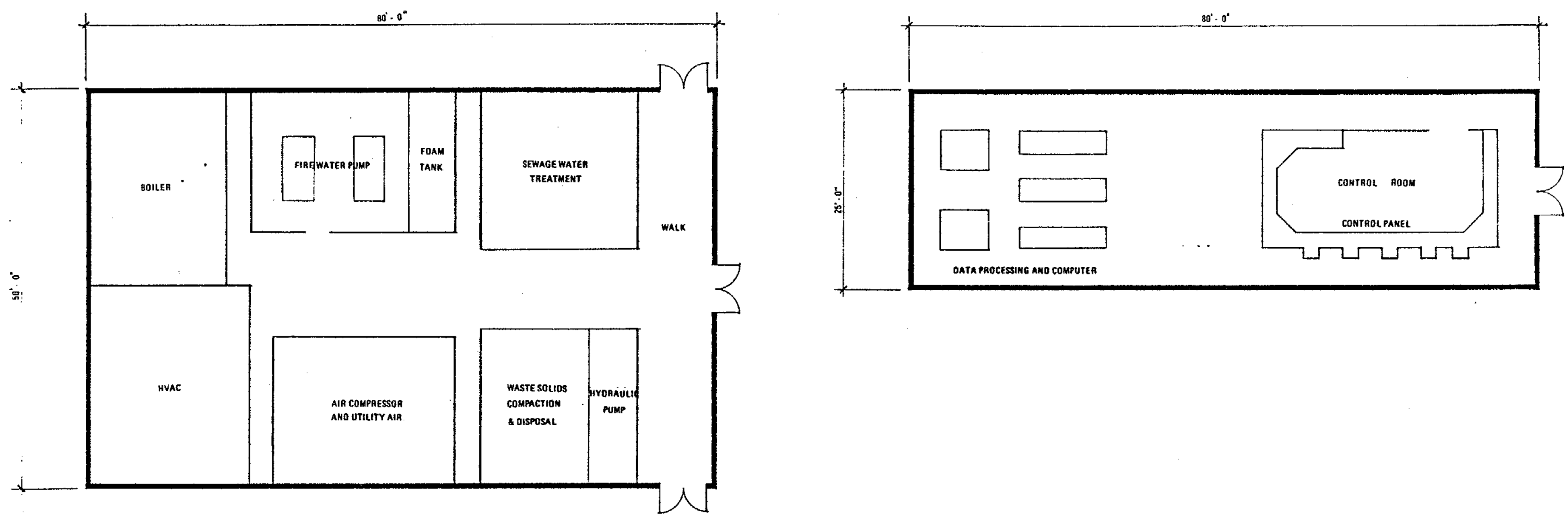
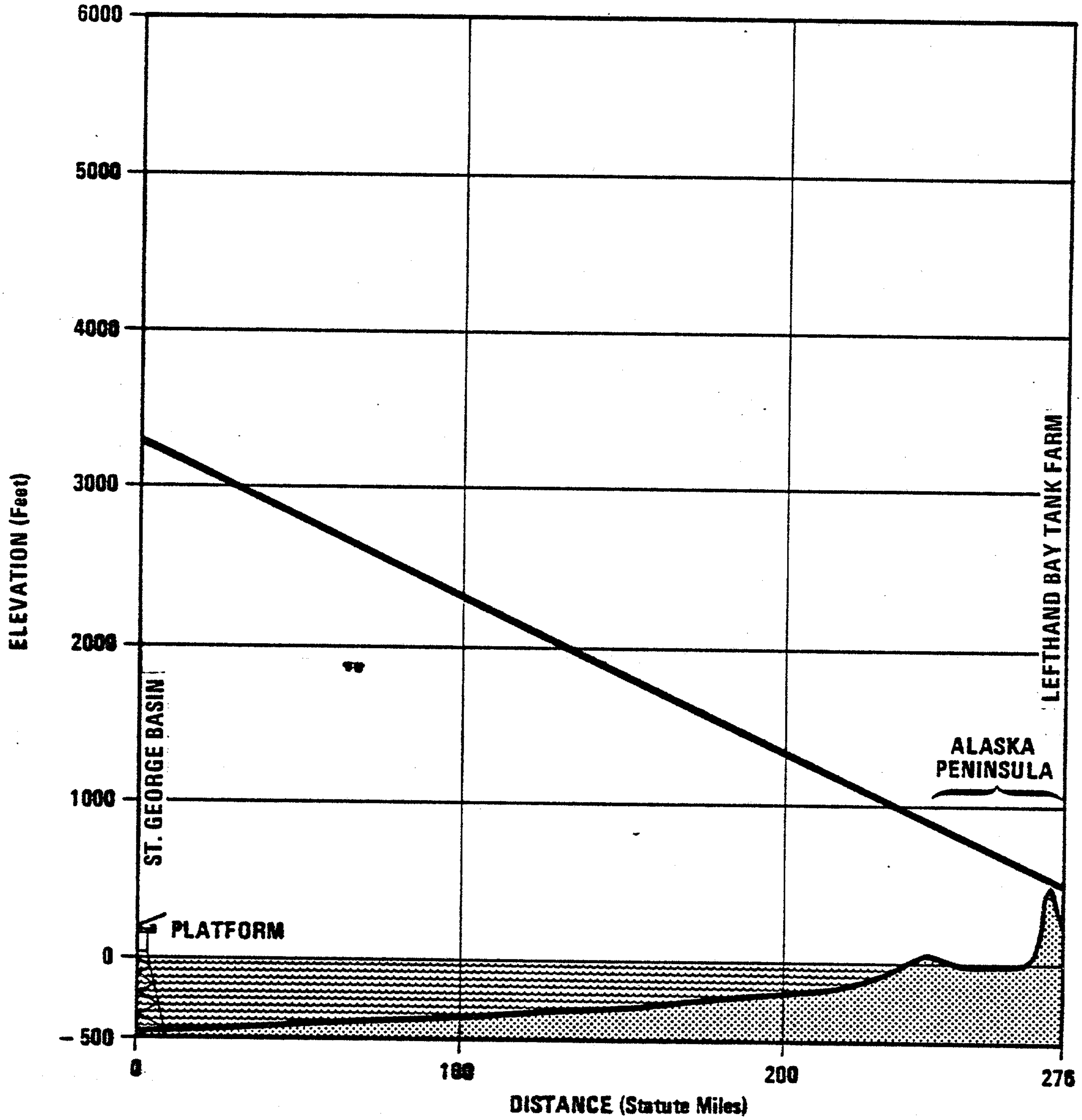


Figure 2-22

Southern Bering Sea Production Systems Study

CONTROL ROOM AND UTILITIES MODULES FOR 100 AND 300 MBPD STEEL STRUCTURE

Job 15154 June 1982

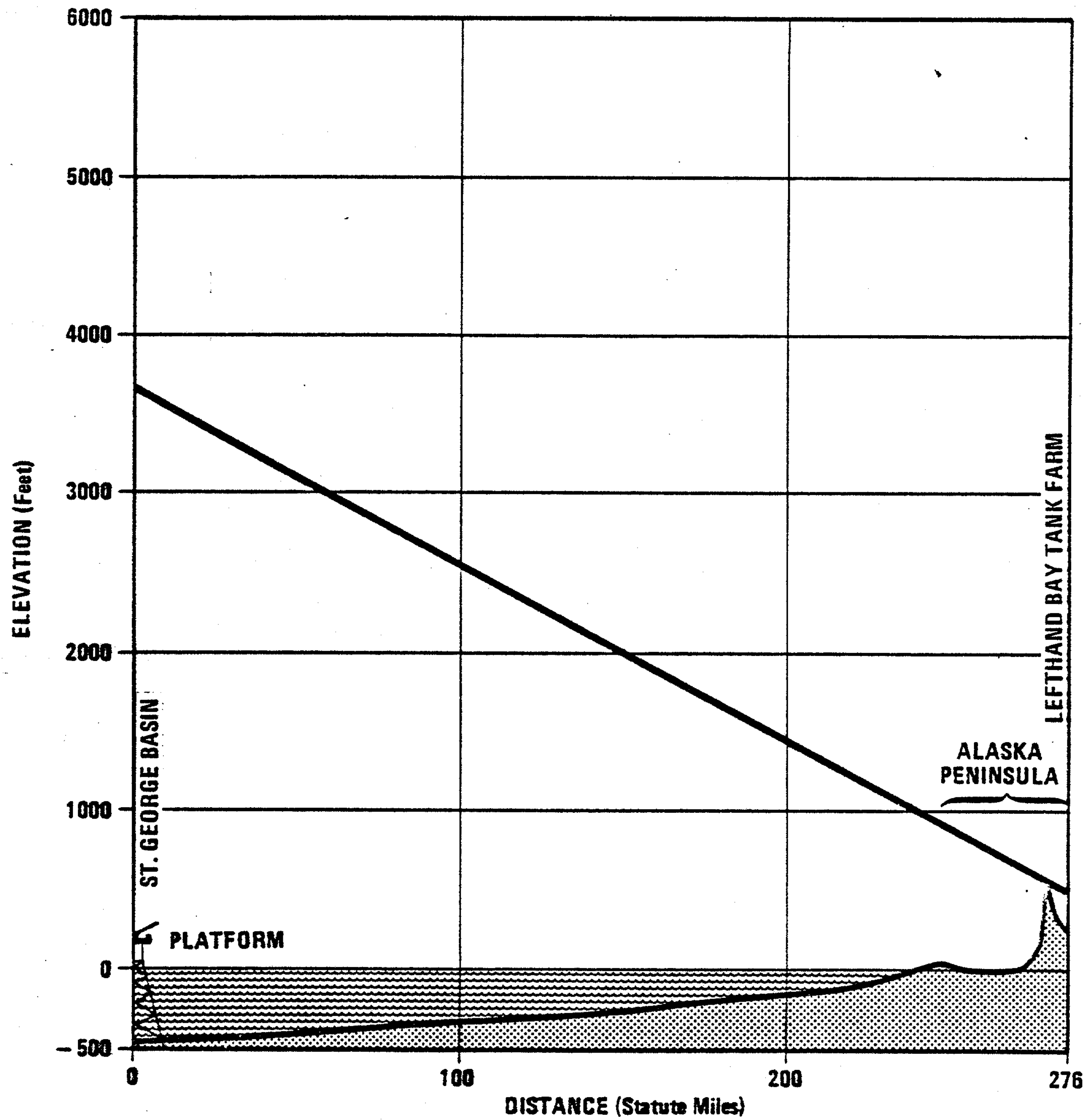


PIPE O. D.	24 INCHES
AMB. TEMP.	35°F
KIN. VISCOSITY	110 CENTISTOKES

Figure 2-23
Southern Bering Sea Production Systems Study
St. George Basin To Lefthand Bay
24" Diameter – 100 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 BECHTEL June 1982



PIPE O. D. – 36 INCHES
AMB. TEMP. – 35°F
KIN. VISCOSITY – 450 CENTISTOKES

Figure 2-24
Southern Bering Sea Production Systems Study
St. George Basin To Lefthand Bay
36" Diameter – 300 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 June 1982

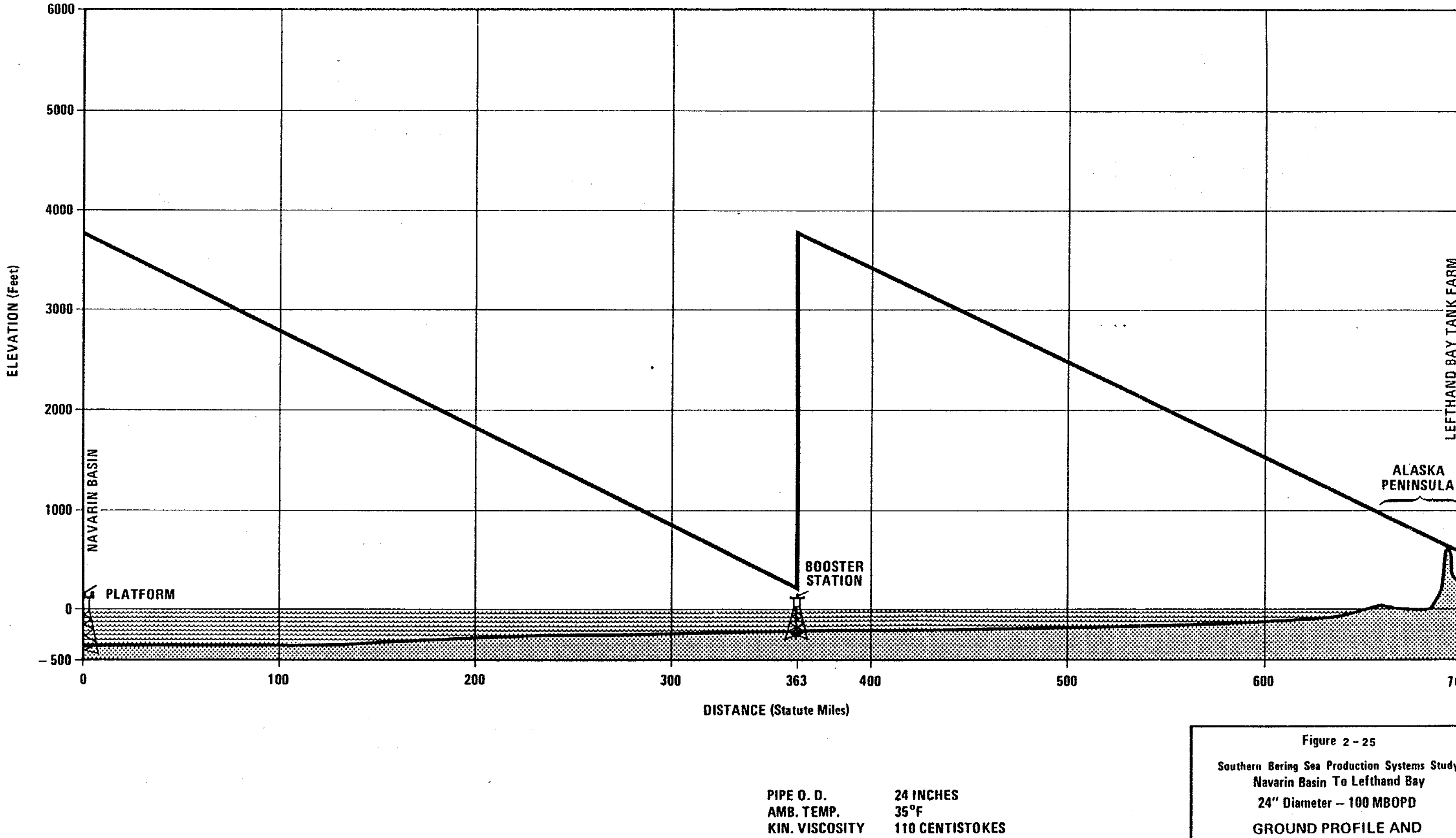


Figure 2 - 25

Southern Bering Sea Production Systems Study
Navarin Basin To Lefthand Bay
24" Diameter – 100 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 June 1982

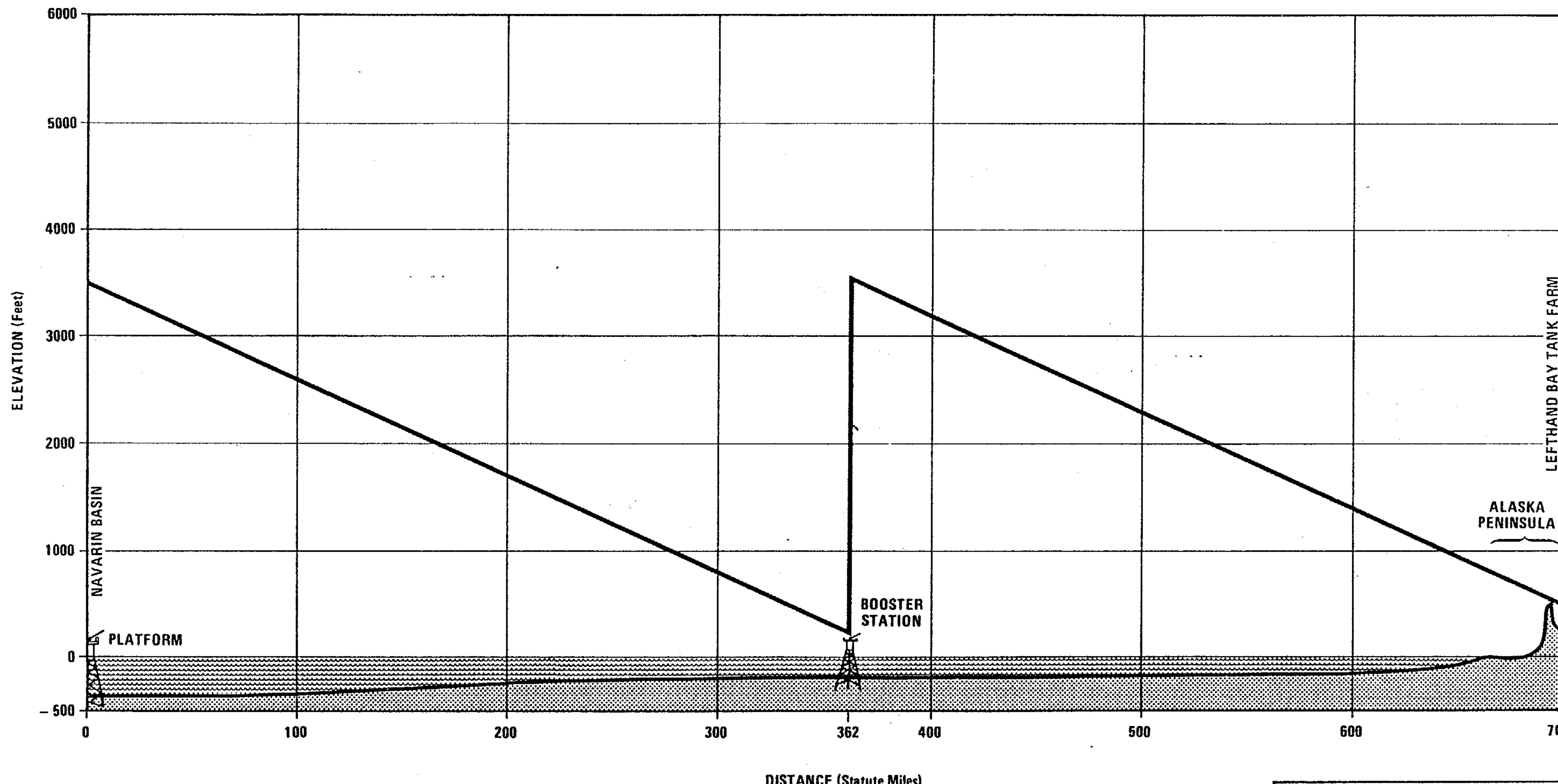


Figure 2-26
Southern Bering Sea Production Systems Study
Navarin Basin To Lefthand Bay
36" Diameter – 300 MBOPD
GROUND PROFILE AND
HYDRAULIC GRADIENT

Job 15154 June 1982

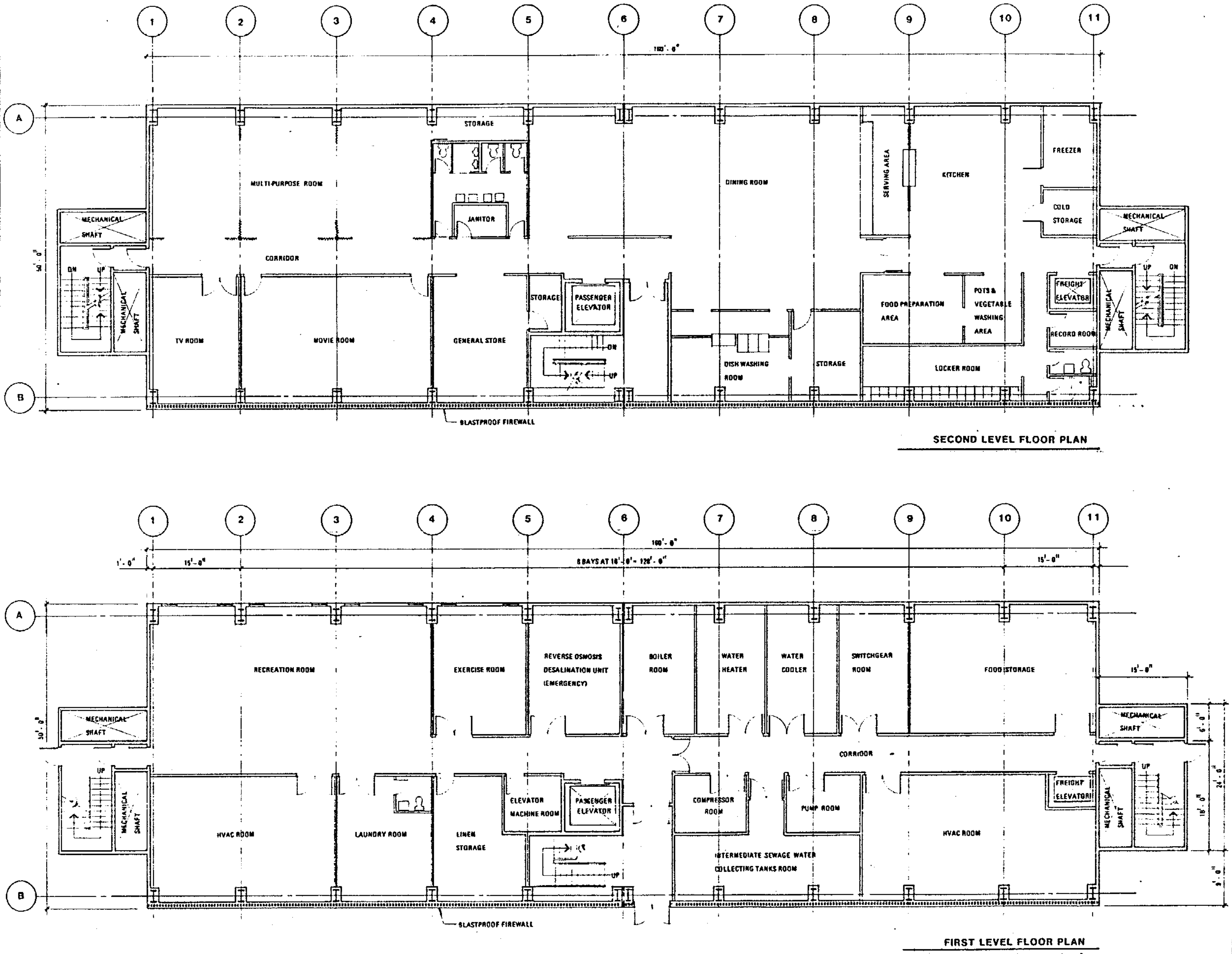


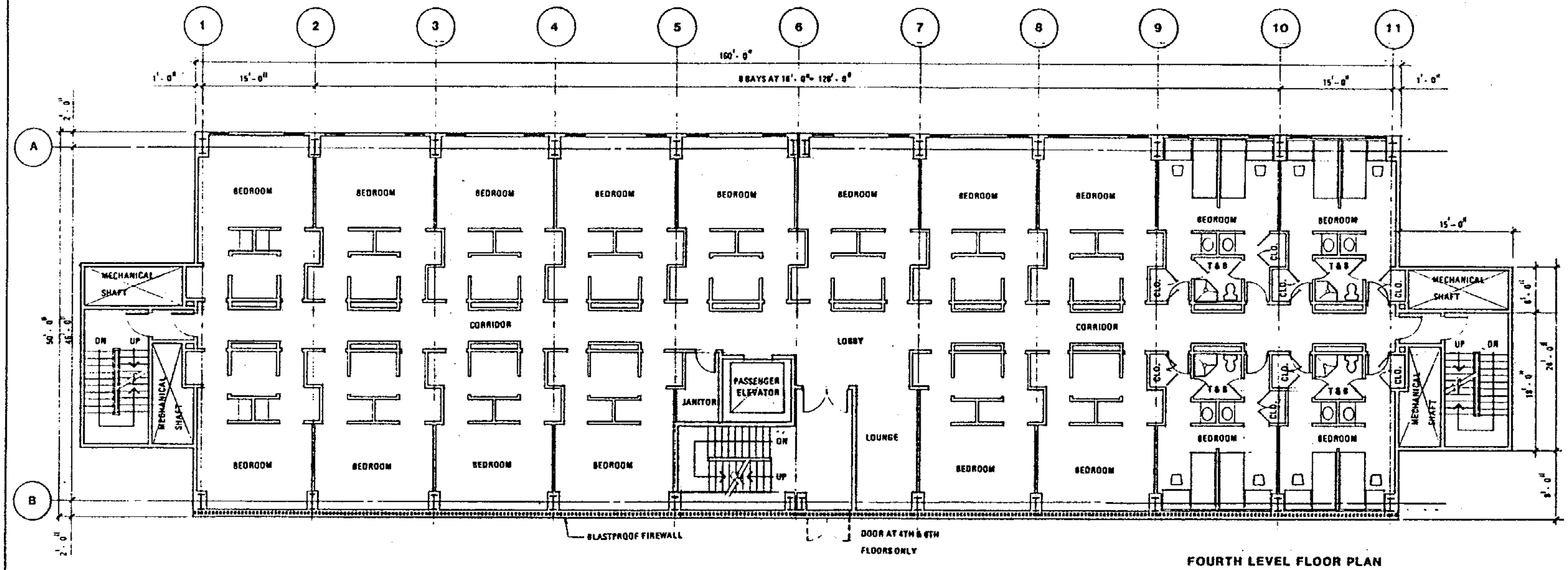
Figure 2-27

Southern Bering Sea Production Systems Study

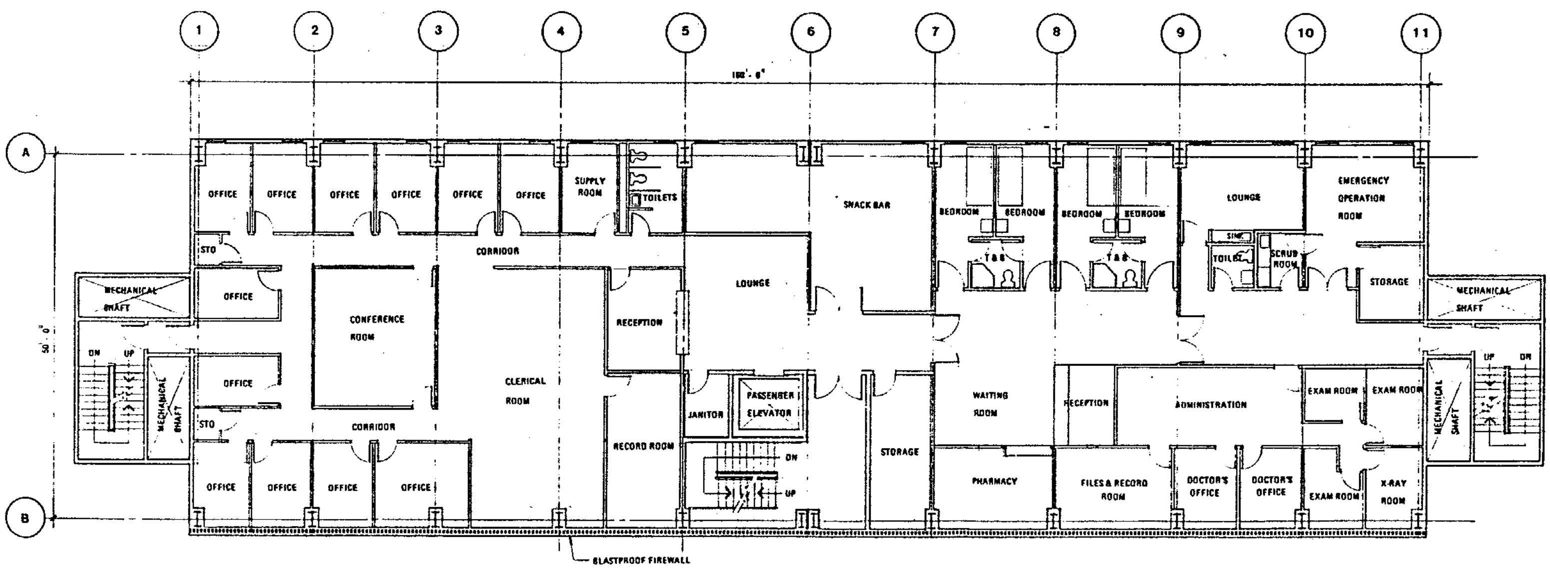
ACCOMODATION MODULES

-Service and Facilities-

Job 15154 June 1982



FOURTH LEVEL FLOOR PLAN
(TYPICAL SLEEPING QUARTERS)



THIRD LEVEL FLOOR PLAN

Figure 2-28
Southern Bering Sea Production Systems Study

ACCOMODATION MODULES

-Office and Sleeping Quarters-

Job 15154 June 1982

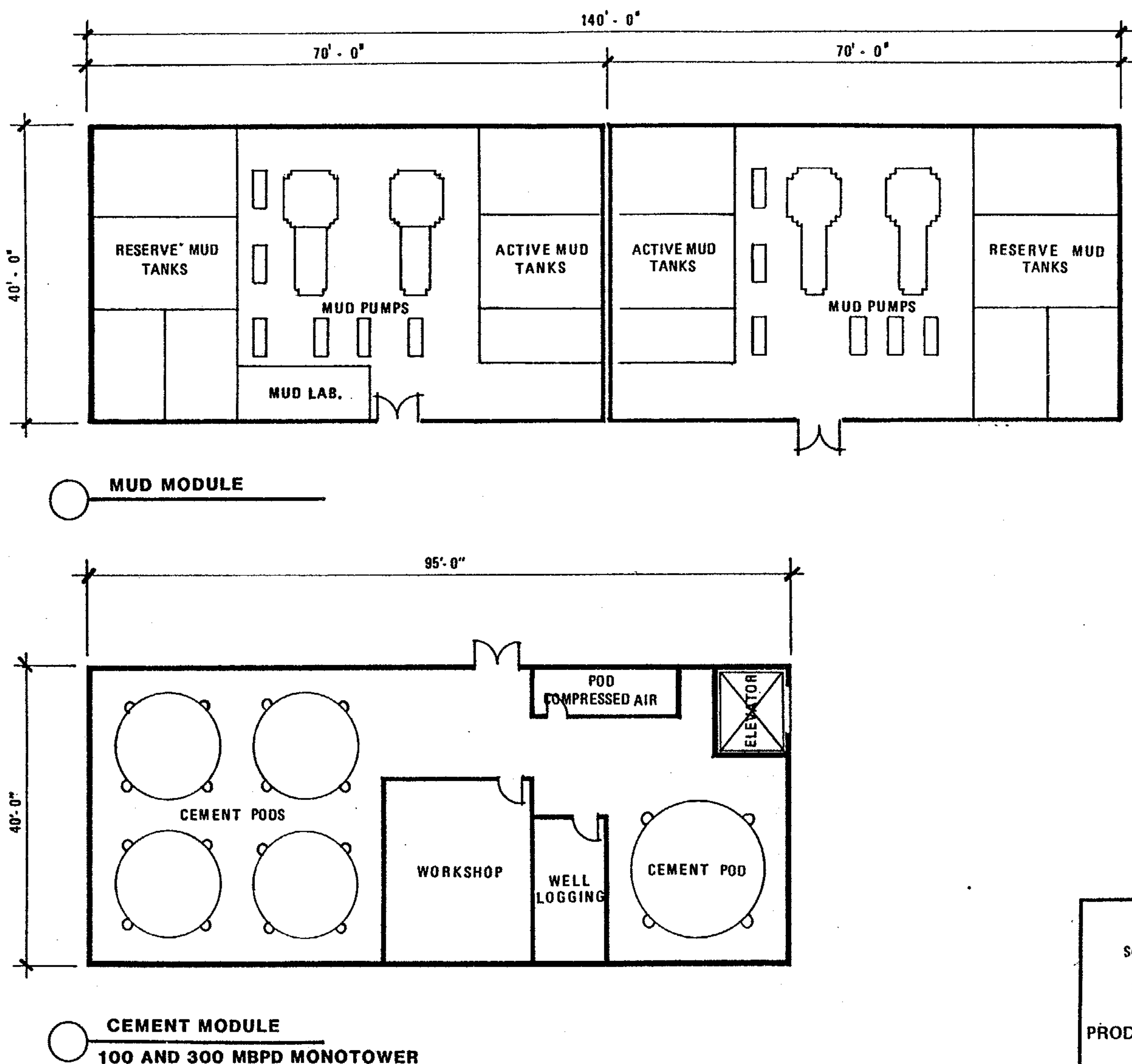
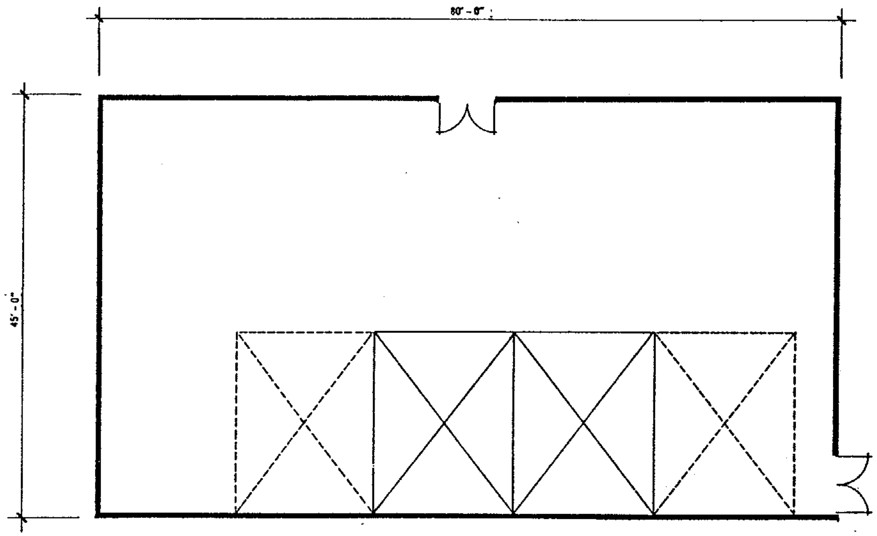


Figure 2-29

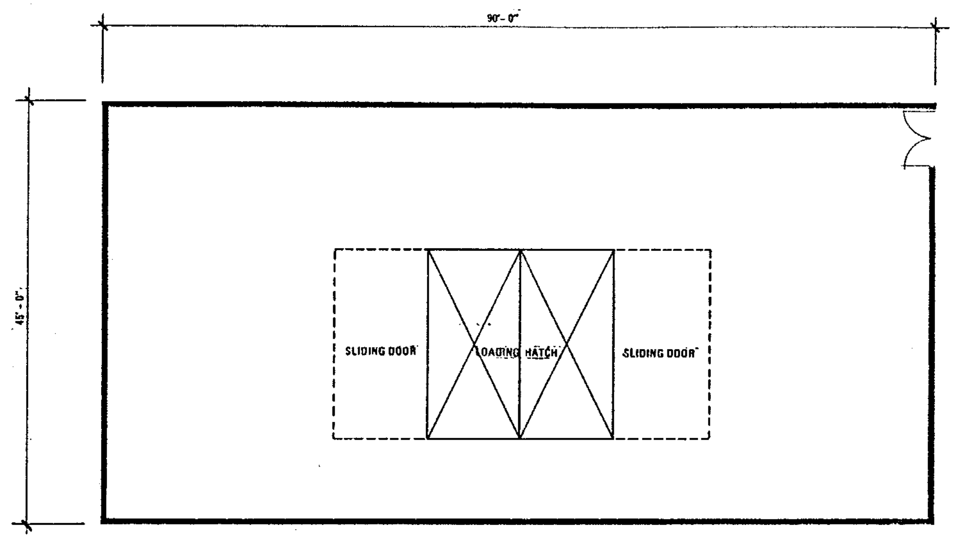
Southern Bering Sea Production Systems Study

MUD AND CEMENT MODULES
FOR 100 & 300 MBPD
PRODUCTION-CONCRETE MONOTOWER

Job 15154 June 1982



STORAGE-1 MODULE



STORAGE-2 MODULE
THE 300 MBPD IS THE SAME EXCEPT
LOADING HATCH LOCATION IS SHIFTED

Figure 2-30

Southern Bering Sea Production Systems Study

STORAGE MODULES FOR 100 & 300 MBPD
PRODUCTION-CONCRETE MONOTOWER

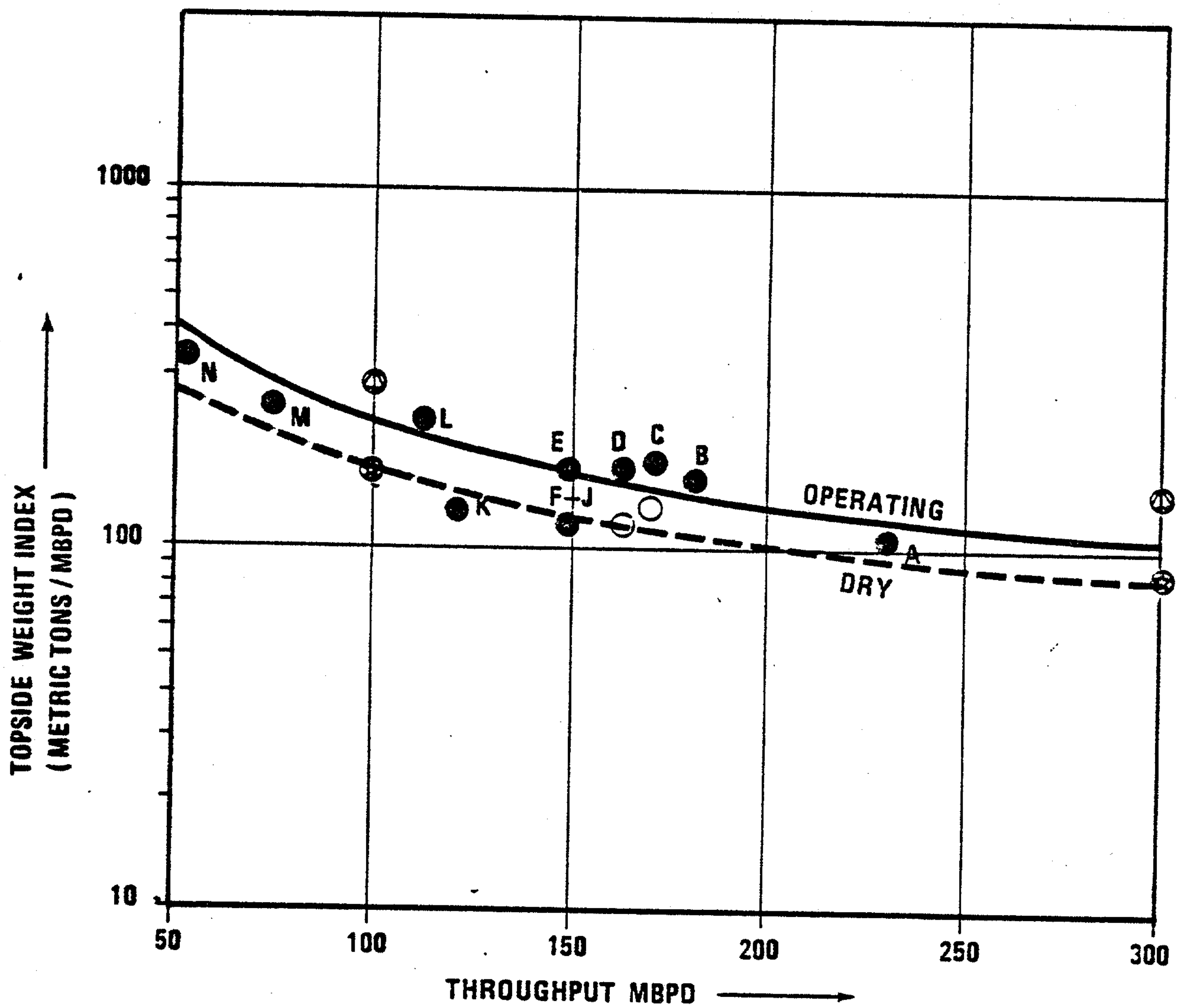
Job 15154

June 1982

FIGURE 2–31

TOPSIDE COMPONENTS – WEIGHTS

MODULE #	WEIGHT (KIPS) / DESCRIPTION	STEEL STRUCTURE				MONOPOD			
		100 MBPD		300 MBPD		100 MBPD		300 MBPD	
		DRY	OPERATING	DRY	OPERATING	DRY	OPERATING	DRY	OPERATING
1	GAS INJECTION	2180	2320	3350	3590	2290	2430	3520	3770
2	FLOWLINES	1000	1100	1200	1320	1000	1100	1200	1320
3	WELL HEAD MODULES (A & B)	2310	2430	2550	2700	1850	1940	2040	2160
4	WATER INJECTION	1130	1350	2500	3070	1070	1280	2370	2910
5	SEPARATION	1980	2730	3030	4470	2410	3570	3850	4700
6	MAINLINE PUMP(S)	970	1010	1870	1960	–	–	–	–
7	BALLAST WATER TREATMENT	–	–	–	–	270	2040	520	3940
8	UTILITIES	1500	2160	2900	4180	1800	2590	3480	5010
9	POWER GENERATION	2680	2700	5180	5220	2680	2700	5180	5220
10	ACCOMMODATION AND HELIDECK	4560	4690	5820	5980	4560	4690	5820	5980
11	DRAW WORK(S) AND DERRICK(S)	1600	5230	1600	5230	800	2610	800	2610
12	MUD PUMPS AND TANKS	1040	6140	1250	7370	730	4090	870	4910
13	STORAGE	900	5360	1080	6430	760	3570	910	4290
14	CEMENT	500	3410	600	4130	420	2290	500	2750
15	EMERGENCY POWER	800	1900	1550	3500	800	1900	1550	3500
16	CONTROL ROOM	1430	1440	2770	2790	1430	1440	2770	2790
17	FLARE	840	840	1600	1600	880	880	1680	1680
18	PIPE RACK	–	4000	–	6000	–	4000	–	4000
	15% CONTINGENCY	3810	7320	5830	10,430	3560	6470	5560	9230
	DECK STRUCTURE	6400	6400	11200	11200	6600	6600	11600	11600
	CRANES	600	600	600	600	600	600	600	600
	TOTAL WEIGHT (KIPS)	36200	63100	56500	91800	34500	56800	54800	83000
	(SHORT TONS)	18100	31600	28300	45900	17300	28400	27400	41500
	TOPSIDE WEIGHT INDEX (METRIC TONS/MBPD)	164	286	85	139	156	257	83	125



KEY

- S. Bering Sea (Wet)
- S. Bering Sea (Dry)
- A THISTLE A
- B NINIAN S
- C VALHALL A
- D MURCHISON
- E FULMAR
- F FORTIES, FA, FB, FC & FD
- J N. CORMORANT
- K NINIAN N
- L MAGNUS
- M MAUREEN
- N HEATHER

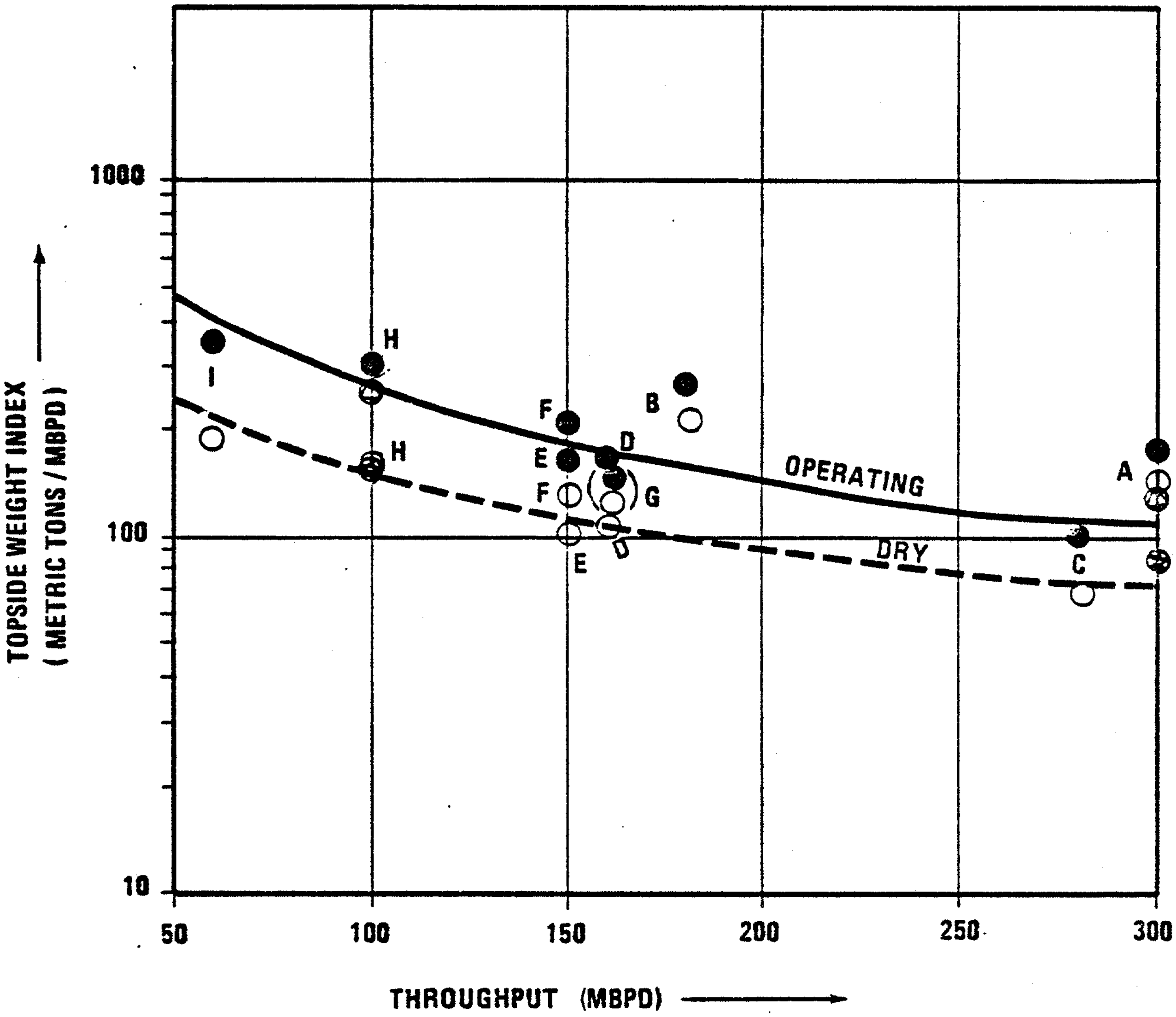
- OPERATING
- DRY

Figure 2-32

Southern Bering Sea Production Systems Study

TOPSIDE WEIGHT INDEX VS CRUDE OIL PRODUCTION RATE STEEL PLATFORMS

Job 15154 BECHTEL June 1982



KEY

- S. Bering Sea (Wet)
- S. Bering Sea (Dry)
- A STATFJORD A
- B STATFJORD B
- C NINIAN C
- D BRENT B
- E DUNLIN A
- F BRENT C
- G BRENT D
- H BERYL A
- I CORMORANT A

- OPERATING
- DRY

Figure 2-33

Southern Bering Sea Production Systems Study

TOPSIDE WEIGHT INDEX VS CRUDE OIL PRODUCTION RATE —CONCRETE PLATFORMS

Job 15154 June 1982

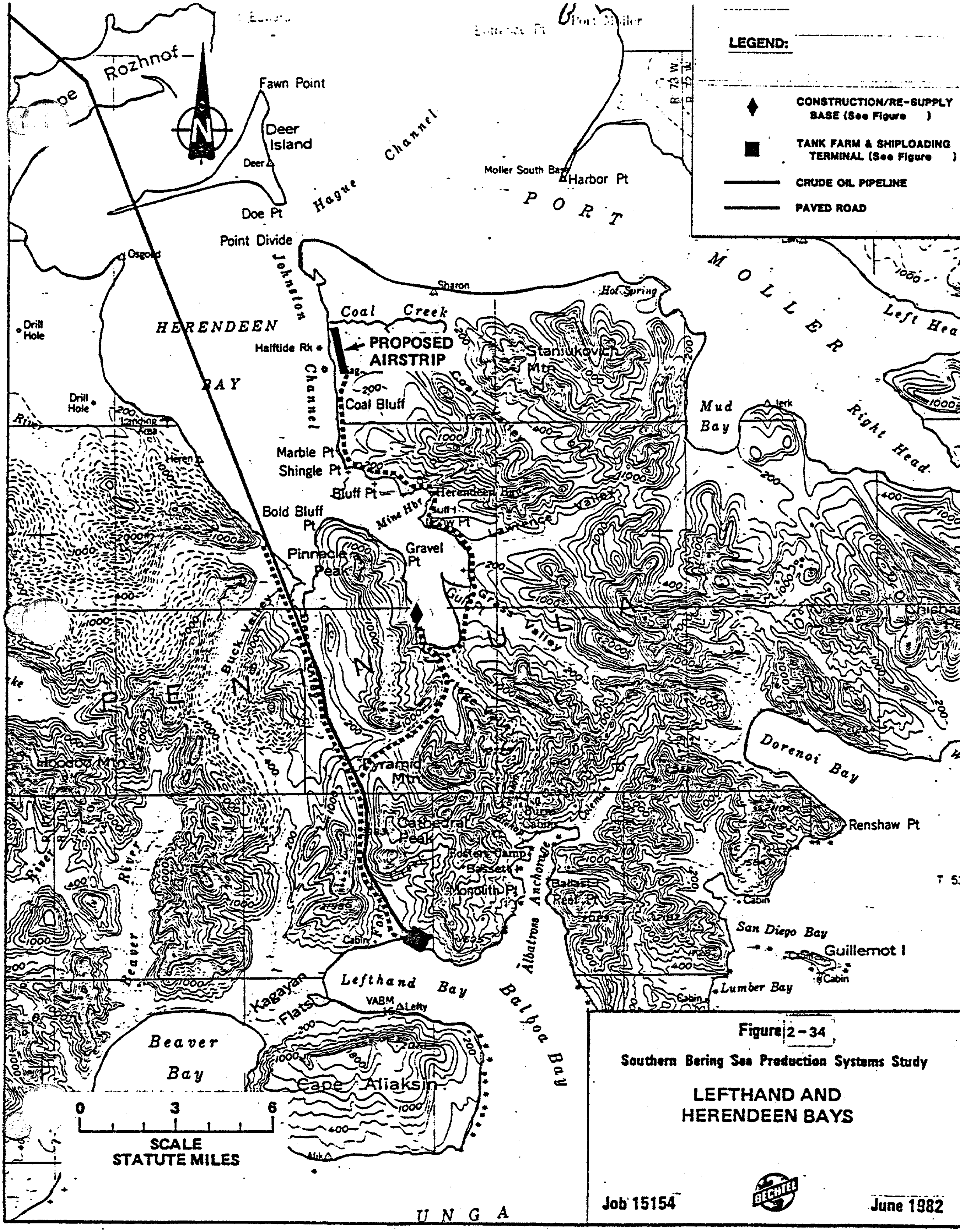


Figure 2-34

Southern Bering Sea Production Systems Study

LEFTHAND AND HERENDEEN BAYS

Job 15154

BECHTEL

June 1982

TABLE OF CONTENTS

SECTION 3.0

LIST OF TABLES

SECTION 3.0

LIST OF FIGURES

SECTION 3.0

LIST OF TABLES

SECTION 3.0

3.0 PLATFORM ALTERNATIVES

3.1 Scope of Work

Prior to the initiation of preliminary designs for drilling and production platforms (documented in Sections 4 and 5), several platform configurations were qualitatively evaluated. Concepts evaluated included the following:

- Steel Pile Supported Structures
  - Eight Leg Template with Skirt Piles
  - Four plus Four Template with Skirt Piles
  - Eight Leg Template with Caisson
  - Self Floating Tower Type with Cluster Piles
- Concrete Gravity Structures
  - Four Leg Condeep Type
  - Condeep Monotower
  - Twin Tower
  - Tripod with Monotower
  - Chevron Monotower
- Floating Production System
  - Semi-Submersible

Not all possible configurations were examined; however, it was felt that by briefly examining and evaluating several of the fundamental options that one steel and one concrete structure could be selected as the most promising alternative for the criteria selected. It is very possible that concepts which were rejected may have future application if the selection criteria is changed.

The floating production system documented in this section is discussed in more detail than the steel or concrete structures. The primary discussion of the fixed platforms is contained in the following two sections.

Cost estimates for the chosen steel system, concrete system, and floating production system are given in Section 6.

3.2 Selection Criteria

3.2.1 Track Record of Concept

As much as possible, proven concepts were favored over concepts or configurations which had not been constructed in hostile environments previously. This provided a good basis for establishing credible preliminary designs quickly, and increased the level of confidence in the associated costs.

3.2.2 Flexibility

The ability to adapt the concept to changes in operational or environmental criteria was an important selection criterion. This is especially true in early feasibility studies, but also for site specific projects. It is imperative that a concept not be overly sensitive in feasibility or cost to relatively minor changes in criteria.

3.2.3 Suitability to Specified Criteria

Since the Southern Bering Sea has three significant environmental threats (storms, earthquakes and ice) plus very large topside, and in the case of concrete gravity platforms, storage requirements, a structure must be configured to deal with these multiple considerations. Some structures may have been very efficient in dealing with one environmental element but poor for others. Concepts were favored that offered a balance even if they performed more poorly for one particular aspect than another concept.

3.3 Steel Structure Alternatives

Several types of steel pile-supported platforms have been qualitatively evaluated for their applicability in the Southern Bering Sea. The structures considered are (1) an eight leg template with one vertical face, (2) a four plus four template, (3) an eight leg template with all faces battered and a caisson to protect the wells, and (4) a four-leg self-floating tower. Additionally, a trussed deck support structure has been compared against a box girder system, and the problems and benefits of diagonal bracing from the deck level to the top of jacket have been reviewed.

3.3.1 Eight Leg Template with Skirt Piles

Figure 3-1 shows schematic elevations of an eight leg template. It has one vertical face with two large diameter vertical legs through which the wells can be run. Braces piercing the water surface are avoided because they are susceptible to damage from ice and they add congestion, which tends to encourage ice jamming. Advantages for this structure are good structural redundancy, ample support points for the deck and the capability to readily accommodate a two-piece deck substructure, easy accommodation of conventional pile driving, and easy adaptation to barge transport and launch at sea. Its major disadvantage is that it is too congested at the water surface to avoid ice jamming. That is, there is insufficient clearance between the legs to allow ice to clear, and it is probable that rubble piles could build around the legs and eventually lead to lateral ice pressure across the plane of the entire structure cutting the water surface. The structure requires supplemental skirt piles to support the heavy deck loads and large overturning moments that are expected.

3.3.2 Four Plus Four Template with Skirt Piles

The four plus four template (see Figure 3-2) has most of the advantages of the previously described eight-leg template, but by eliminating the corner legs through the ice and air zone it avoids the high potential for ice jamming. Of course, eliminating the upper portions of the corner legs has the negative effects of reducing the number of legs supporting the deck and reducing the structural redundancy at the top of the structure.

3.3.3 Eight-Leg Template with Well Caisson

Another alternative is an eight-leg template with a caisson at the top of the structure to protect the wells through the ice and air zone and where wave kinematics are most severe. This structure is shown in Figure 3-3. Since the wells do not run through the legs, all faces of the structure can be battered. Additionally, a relatively conventional deck arrangement is possible because the location of the caisson is variable to some degree. The major disadvantages of this structure are complex framing between the jacket and caisson and severe congestion at the water surface. This latter characteristic creates a high potential for ice jamming, and the large-diameter caisson necessary to accommodate the wells will experience relatively large hydrodynamic forces.

3.3.4 Self-Floating Tower Type

The last option considered is a four-leg tower (Figure 3-4). Two vertical legs can be used to contain the wells; the other two legs are battered in one direction to help spread the base of the tower. A logical foundation for this structure is clustered piles at each leg. The legs must be spread far apart in order to avoid excessive flexibility and high overturning loads to the foundation. This minimizes the potential for ice jamming, but it necessitates using supplemental braces to support the very long deck girders, and leads to self-flotation as the most promising transportation method. Transportation by self-flotation is thought to be somewhat more risky than by barge. Another disadvantage of this structure is that the deck arrangement cannot be optimum from a safety standpoint. Since the two vertical legs must be spread far apart, the wells are grouped at two corners on a long side of the deck. A safer deck arrangement locates the wells along a short side of the deck, so that the deck can be stretched away from the wellheads to maximize the distance between the wellheads and other sensitive modules such as quarters.

3.3.5 Deck Structure Alternatives

A substantial deck structure is required to support the heavy payloads that are anticipated. Furthermore, the deck structure must be stiff in bending to complete a moment frame at the top of the structure, if sway braces are to be avoided. Configurations without sway braces are preferred because they minimize the potential for ice jamming.

Figures 3-5, 3-6 and 3-7 show three possible configurations for a deck support structure. A deep truss could provide a reasonably high strength to weight ratio. Its major disadvantage is its depth, estimated to be in excess of 30 feet. Either a very high deck elevation or a lower truss chord strengthened to tolerate wave and ice impacts are implied. A box girder, although probably somewhat heavier than a deep truss, has several advantages. Its required depth is less, probably in the range of 15 to 20 feet. Additionally, it is relatively insensitive to the location of equipment module reactions points, is torsionally stiff, and offers liquid storage potential. The third option is either a shallow truss or girder supported vertically by legs and K-braces. From the standpoint of the decks, this is a good solution because it can be relatively light and shallow without large deflections. However, it has already been noted that the k-braces are susceptible to damage from ice slam and they tend to congest the structure at the water surface, causing ice jamming and increased lateral loads. It is estimated that the braces need be roughly 7-feet in diameter just to resist the bending which could be induced by ice slamming into them at midspan.

3.3.6 Conclusions

To limit the scope of subsequent work, one jacket type and deck support structure type were selected to be carried through preliminary designs at each of the three study basins. The four plus four template was selected because it has good redundancy yet is fairly "clean" through the ice zone, minimizing ice loads and ice jamming. Additionally, the four plus four template can be transported and installed using reasonably conventional procedures. This jacket demands either deep trusses or box girders at the deck to avoid excessive deck deflections and large bending moments in the legs. The box girder scheme was selected because it is shallower, offers torsional stiffness for redundancy and is insensitive to module reactions points.

It should not be construed that the selected structural scheme is the optimum at each basin. Rather, the scheme provides a common basis for developing consistent approximate cost estimates for a structure which has characteristics believed generally applicable for the Southern Bering Sea. For instance, at the Navarin Basin, it is assumed that ice floes and ridges may bridge between platform legs regardless of their spacing. In this case, the "clean" profile of the four plus four template may not offer an obvious advantage in terms of total lateral load. The eight-leg template may be equally as feasible, plus it has more support points for the deck.

3.4 Concrete Gravity Structure Alternatives

Concrete Gravity Base Structures (GBS) have been used for offshore drilling production and storage in the North Sea since 1973 (see Table 3-1). The use of similar structures in the three basins examined in this study presents additional challenges primarily due to the weaker foundation soils, the presence of sea ice and the relatively higher seismicity of the area.

Five different concrete GBS alternatives were considered in this study. The main difference between the various alternatives is in the number of shafts and their layout. Following is a summary of the main features characterizing each of the alternatives considered.

3.4.1 Four Legged

The tower is composed of four shafts extending between the gravity base and the bottom of the deck (Figure 3-8). The shafts are tapered and describe in plan a rectangular geometry which provides a convenient topside support. The living quarters on the deck can be conveniently separated from the drilling and production facilities by means of a concrete blast wall. Other advantages associated with the four-legged configuration are the existence of structural redundancy and good stability afloat. The latter is due to the relatively increased water plane moment of inertia which contributes to the overall floating stability. Furthermore, the four shafts enable a convenient separation of the drilling production and utility equipment. A possible disadvantage is due to the increased likelihood of ice "jamming" among the shafts. This problem can be, to a large degree, minimized if the net spacing between the shaft is kept above 100 feet. The four legged structure was the selected alternative in this study.

3.4.2 Monotower

The tower is composed of a single shaft extending between the gravity base and the bottom of the deck (Figure 3-9). This structure is expected to perform well in resisting ice loads; however, it doesn't maintain any redundancy, and thus may be exposed to more severe consequences due to accidental damage. Furthermore, the deck support is not efficient due to the cantilever geometry. The structure's floating stability is relatively small and may require auxiliary means such as heavy ballasting or temporary buoyancy during the transportation and installation phases.

3.4.3 Twin Tower

This alternative (Figure 3-10) is, perhaps, a midway solution between the previous two alternatives. It maintains some limited structural redundancy, however, the deck support is not very efficient due to the partial cantilever geometry. The floating stability of this alternative is relatively small due to a limited water plane moment of inertia about the minor axis. Hence, auxiliary stabililty means similar to the monotower alternative may be required.

3.4.4 Tripod with Monotower

The tower of this structure is a combination of three inclined columns supporting a single shaft vertical tower (Figure 3-11). This alternative may be suitable to deep water sites, primarily due to the reduced wave loads on the base caisson. It is expected to perform well in resisting ice loads; however, at the present time it constitutes an unproven construction method. As with to the monotower alternative, the deck support is inefficient due to its cantilever geometry. Furthermore, it is difficult to provide a structural separation between the living quarters and the drilling production equipment.

3.4.5 Chevron Concept

This structural alternative differs from all the other alternatives in its base geometry. The base is composed of three wing flat slab framing into three perpendicular walls with bottle like columns at each end (Figure 3-12).

The tower is composed of a single vertical circular shaft which provides the structure with good ice load resistance capability. The concept is particularly suitable for deep water sites, where no major crude oil storage is required. On the other hand, the construction of this structure calls for the use of unproven methods which could make this alternative economically unfeasible. Other disadvantages present in this concept are the low torsional resistance capacity, the absence of storage space, and the inefficient deck support system.

3.4.6 Conclusions

In order to limit the scope of subsequent work, one gravity structure type was selected to be carried through preliminary designs at each of the three lease sale areas. The four leg Condeep style was selected primarily because of the redundancy of four legs, potential for storage, good deck support, plus its proven track record (Statfjord B and C). Furthermore, the primary potential benefits of utilizing fewer main shafats, ile., lower ice forces or concrete savings, were not thought to be significant. This is due to the presumed predominance of seismic or oceanographic criteria for the vast majority of cases in the Southern Bering Sea.

3.5 Floating Production Systems

3.5.1 Introduction

A candidate floating production system is discussed in somewhat greater detail than the steel or concete alternatives were discussed in Sections 3.3 and 3.4. This is because floating production systems were not pursued through a preliminary design as were the steel or concrete platforms as detailed in Sections 4 and 5. The primary limitation of the floating systems is thought to be the lack of a field proven floating system which could provide a base for continuous operations for the anticipated high production rates. There are untested floating concepts which have been proposed such as that shown in Figure 3-13 which could resist the anticipated ice forces, and also provide for a significant payload. The current study did not pursue these concepts.

We believe that proven floating production systems do have a possible application as outlined in this section, even though they do not meet all the specified criteria.

3.5.2 Scope of Work

The intent of this study was to evaluate the feasibility of a representative floating production system for use in the Southern Bering Sea. The scope of the study is focused on field production rates (per vessel) of (50,000) BPD or less. The study, therefore, lends itself in general towards evaluation of production alternatives for marginal fields, early production and/or extended production testing.

Given the study scope, there are several potentially feasible floating production systems. It was felt, however, that the main effort be directed as much as possible towards systems consisting of state-of-practice, field proven components. Based upon these requirements, there were basically two types of production alternatives considered: semi-submersible platforms and ship-shape platforms. In general, either alternative is a viable one. In a harsh environment such as the Bering Sea, however, semi-submersibles are field-proven. A ship-shape production vessel has yet to be designed and moored in a severe environment. Whereas a semi-submersible has a relatively small waterplane area compared to its displacement, a ship-shape vessel has a relatively large waterplane area compared to its displacement~~, a ship-shape vessel has a relatively large waterplane area,~~ resulting in ~~the~~ greater susceptibility to wave-induced motions and high mooring forces. In an effort to focus the study on field proven components, semisubmersibles were selected for the candidate production platforms.

The following production systems are evaluated in the study:

- Purpose Built Semi
  4000 to 7000 Ton Payload
  25,000 to 50,000 BPD Production
- Converted Drilling Semi/Ice Strengthened
  2000 to 3000 Ton Payload
  Less Than 25,000 BPD Production
- Converted Drilling Semi/Nonmodified
  2000 to 3000 Ton Payload
  Less Than 25,000 BPD Production

The following system components are also identified and evaluated:

- Product Offloading and Storage Systems
  (SPM, storage tanker)
- Semi Mooring System
- Subsea Template and Marine Riser

The products of this study include an evaluation of the various production system components in an environment typical of the Southern Bering Sea. This includes consideration of structural ice strengthening of all affected components and also an evaluation of total system disconnection and site abandonment in the event of sea ice intrusion exceeding the system design conditions. Much of the evaluation is in the form of design recommendations.

These recommendations are based in part upon information gathered from meetings and discussions with drilling contractors with experience in ice environments (example: Cook Inlet) and also from meetings with product vendors.

In addition to system design recommendations, estimates of system costs and schedules are included in Section 6. Although these will be approximate costs they should facilitate in planning for development of future lease sales.

Considering that some of the required components and/or modifications and their limitations are either not state-of-practice or not known, recommendations for future work are also included throughout the study.

3.5.3 Application

Floating production systems can have decided advantages over fixed platforms for both moderate and severe environment field development. General advantages include:

Early Production

Marginal Field Development

Early Reservoir Evaluation

Cost Reduction

Re-usable

Minimal seismic and soil constraints

In a severe environment such as the Bering Sea, principal advantages of floating production systems include:

Mobility for Resiting or Site-Abandonment

Design Need Not Resist Maximum Expected Loading Conditions

In the Southern Bering Sea, ice floes and wave-ice interaction can produce excessive loads. However, especially in the southern basins, seasons of ice coverage (say three oktas and greater) can be quite short and summer seasons* quite long, even one or more years.

3.5.4 Semi-Submersible Production Systems

The proposed semi-submersible production system would consist of seafloor wells produced through a subsea template and possibly satellite wells connected by flowlines to the template. Template wells allow wireline workover to be performed. The gathering manifold would be located on the subsea template and connected via a production riser to the semi. All seafloor equipment would be controlled from the surface. Process facilities would be located on the deck of the semi. Processed oil would be pumped through a sales line back to the seafloor into the storage and/or offloading system. In order to minimize required deck payloads, it is assumed that the associated gas produced would be flared and treated produced water would be pumped overboard.

As a reference, semi-submersible production facilities are presently on line and producing in the Argyll and Buchan Fields in the North Sea.

*Summer season is defined as the period of time when total ice concentration is less than or equal to a specified concentration.

A schematic diagram of the candidate floating production system is shown in Figure 3-14.

3.5.5 Evaluation of System Components

Each of the basic components of the semi-submersible floating production system will be discussed and evaluated in terms of its adaptability to an environment typical of the Southern Bering Sea. The main focus is on ice strengthening and design for quick disconnect.

3.5.5.1 Semi-submersible

Converted Drilling Semi

As was stated earlier, there are two types of converted drilling semis considered in this study. One is assumed to be modified for sea ice conditions (example: Baltic Class 1-C Design Criteria) and the other is assumed to be nonmodified. Each are typical of the SEDCO 700 series semi-submersibles with production capacities of about 25,000 BPD or less.

A typical profile and deck layout for this class converted semi is shown in Figures 3-15, 3-16, and 3-17.

Baltic Class 1-C design criteria require strength to withstand 75 psi sea ice pressure, 24 inches thick. Modifications can be provided for either transit or transit and drilling (production) draft. The rationale for providing transit draft modifications only would be that of an insurance measure. The operator would not plan to work in sea ice conditions, but if caught in such conditions, the structure could safely abandon the site. Structural modifications for transit draft ice strengthening include:

- Increase plate at the bottom of the columns.
- Increase plate thickness on the top of pontoons.
- Add stiffening girders to pontoons.
- Add plate to pontoon bow sections.

In order to provide ice strength at production draft, the following additional modifications need to be made:

- Add longitudinal plate stiffeners to vertical members.
- Add deep ring girders and longitudinal stiffeners to columns.

In a sheet ice environment, the vertical structural bracing is the weakest structural link. These members are typically 5 feet in diameter and 100 feet long. Invading ice floes can create local failure and cause bending at the joints. In addition, wave-ice interaction can produce local ice loads of sufficient magnitude to knock out these vertical bracing members. Additional measures can be taken to reduce the risk of such damage. These include the incorporation of an "ice fence" to elastically absorb ice impact loads. SEDCO has developed an ice fence design to be placed around the perimeter of the semi. It would be constructed of 1-1/2 inch bridge strand and the design is estimated to withstand 50' x 50' x 6' ice floe thickness producing loads up to 800 to 900 kips with floe velocities of 1 to 2 knots.

Another measure to reduce the likelihood of local structural damage due to sea ice involves the utilization of workboats to divert the ice floes around the structure. The maximum ice coverage for such an operation is probably 3 oktas. Greater coverage than this would require failing the invading ice rather than diverting it around the structure.

Referring to the required structural modifications of ice-strengthening at transit and production draft, SEDCO has estimated the amount of steel to perform such modifications to be about 600 tons. This weight includes the estimated weight of the ice fence. This additional weight will directly reduce the amount of allowable deck payload; however, there is still ample capacity for the production systems and throughputs considered in this study.

For the case of the non-modified semi, it is assumed that operations will be continued until there is risk of ice-structure interaction. At this time, the riser and moorings will be disconnected and the site abandoned.

Based upon knowledge of particular reservoir characteristics as well as expected ice conditions, economic analyses can be performed relating the additional cost for ice strengthening to the anticipated additional revenue due to extended on-site production time.

Purpose-Built Semi

As stated earlier, the purpose-built semi described in this study would have a deck load capacity of 4500 to 7000 tons and a production capacity of up to 50,000 BPD. To accommodate such capacities, this purpose-built semi would be larger than any semi built to date. However, accompanying the recent interest in sub-arctic field production, a number of designs for all-weather, ice class semis have been proposed.

One such semi design is shown in Figure 3-18. This is the recently proposed Santa Fe Sub-Arctic semi. The hulls and columns are ice strengthened. In addition, the design includes a winterization system which includes temperature controls to prevent ice accumulation on the caissons and underside of the upper hull.

Other companies involved in the development of ice-class semis are SEDCO, Global Marine, Zapata and Mitsui.

One characteristic of purpose-built ice-class semis is the substantial reduction or elimination of vertical framing. These are the weakest structural links in conventional semis under ice loading conditions. The required structural stiffness (racking, hogging, sagging) is obtained through stronger and stiffer pontoons, stronger and stiffer columns, and incorporation of deep-girder and plate-stiffened or shallow truss deck design. The bottom of the deck support framing should be above the level of potential ice impact. The hulls are of double skin construction.

These purpose built semis, as discussed in this study, have two main advantages: greater production (and possibly storage) capacity and greater ability to withstand ice forces. The question is whether these advantages outweigh the additional cost and required construction times of these structures. Such evaluations must be based upon estimated reservoir characteristics and reliability considerations.

3.5.5.2 Semi-submersible Mooring Systems

In a sea ice environment, the semi mooring system is the second weakest structural link in a conventional semi production system. To increase the capacity of the conventional drag anchor mooring system, anchor piles may be used to give additional restoring force to the existing mooring system. In addition, the number and/or diameter of the anchor lines may be increased.

A proposed mooring system for a purpose-built semi includes a 12-point mooring arrangement with all-chain mooring lines. Such is the design for the Santa Fe Sub-Arctic semi. SEDCO has developed a purpose-built ice class semi-design (4,500 ton deck payload capacity) which utilizes a similar arrangement with anchor piles.

In the event that ice conditions exceed the limiting operational conditions for the floating production system, the mooring system should be equipped with state-of-practice remote controlled selective quick disconnect moorings. By permitting instantaneous disconnection of the mooring lines, production time can be extended by avoiding premature site abandonment. This time saved can allow for work boats to break up ice floes and permit production to continue. Considering the daily rates for various production support systems, the economics of such a system is easily realized. These also provide an important safety measure as ice floes can move rapidly and prohibit conventional anchor handling. There are presently 120 rig anchor release units in use throughout the world.

3.5.5.3 Storage and Offloading Systems

It is assumed that for an early production system or the production of a marginal field that the candidate offshore storage and offloading systems would involve either a dedicated storage tanker moored to a single-point mooring with shuttle tanker offloading or a "tight-line" batch operation with no oil stored on location and shuttle tankers offloading directly from an SPM. Considering the remote location as well as sea ice and meterological conditions, however, it is probable that the dedicated storage tanker system would prove to be more attractive due to the substantial reduction in downtimes.

The dedicated storage tanker system involves the mooring of the tanker to the SPM via a rigid yoke. An example of such a system design for a severe environment (North Sea) is Shell/Esso's Fulmar Field production system. Shuttle tankers may be moored either side-by-side or in tandem with the storage tanker. In severe environments, tandem mooring is less sensitive to storm conditions.

The selection of a particular single-point mooring system for development of the Southern Bering Sea will be strongly affected by the consideration of sea ice and superstructure icing that are prevalent in the area. Terminals that could withstand both severe wave and ice conditions while keeping oil transfer hoses well above the water where they would be unaffected by the ice conditions would lead to very massive and expensive structures. It is therefore possible to take advantage of seasonal terminal operations. The mooring should be designed to withstand local ice loads as an insurance measure; however, when sea ice invades the site to the extent that it is not easily deflected around the mooring, a design is required that provides a means for either submerging the system below the ice or of releasing some or all of the SPM and towing it to safety.

Submergence System

There are a number of possible designs for a submerged SPM. Systems have been considered that are either fully lowered to the seafloor or partially submerged to some point of buoyant equilibrium.

A fully submergible design could be a modified SALM design. At the appropriate time, the yoke would be disconnected from the mooring and towed in by the storage tanker. Once the yoke is detached, the buoyant section of the SALM would be flooded and the structure lowered to the seafloor. The lower universal joint between the base and the leg would be designed to accommodate angles of up to 90 degrees. A small marker buoy would be attached with a pennant line to the submerged structure for a location marker. The re-float operation would be diver-assisted.

Another submergible SPM concept designed specifically for the Bering Sea was developed by IMODCO. This system is known as the Chain Articulated Tower (CAT). It is basically a hybrid between SALM and a CALM system. The catenary chains used to resist horizontal forces on the CAT terminal also allow it to be submerged below the depth of potential ice cover and not cause it to impact on the seafloor which may cause damage. Partial flooding of the buoyancy compartments causes a net downward load which causes the structure to tilt over and submerge. The downward motion is stopped when sufficient chain has been deposited on the seafloor so that the CAT's buoyancy matches its submerged weight. The yoke is designed to counterbalance itself and lock to the storage tanker. This allows the yoke to be easily transported by the storage tanker at the time of site abandonment. Schematic diagrams of the CAT system are shown in Figures 3-19, 3-20, 3-21, and 3-22.

Retrievable SPM System

The single anchor leg mooring (SALM) system is best suited as a retrievable SPM design basically due to the minimum amount of complexity in its design. In fact, a retrievable SALM is essentially a state-of-practice design. The retrieval scheme is basically the reverse of the installation scheme. Either a gravity or pile-founded base would be pre-installed on the seafloor, and the lower universal joint on the riser leg would be attached mechanically. At the time of retrieval, this U-joint would be unlatched hydraulically. The system would then be towed to the shore. One important advantage of a retrievable SPM system over a submergible system is the ability to perform onshore maintenance.

The various single point mooring designs discussed here are the least field-proven components of the floating production systems evaluated in this study. We have assumed one year of pre-engineering work to provide acceptable confidence levels for the development of an effective and economical design.

3.5.5.4 Marine Riser and Subsea Production Systems

The design of the marine riser and subsea production system for use in the Southern Bering Sea is basically state-of-practice. Since no icebergs are present there is no risk of damage to subsea equipment due to ice impact. As a means of protection of the riser from the sea ice, one possible solution is the incorporation of a steel leg or caisson through which the approximately 30 feet in diameter. Another possibility, as mentioned earlier, is the use of the "ice fence" concept to elastically absorb the impact of ice floes.

If the environmental conditions exceed the design conditions for the floating production system, it is simple and proven technology to perform a quick-disconnect of the riser. This involves first purging of all flowlines. The riser tension would then be released to a minimum level and the riser would be remotely disconnected just above the subsea manifold. The riser would then be stroked to avoid impact with subsea equipment. The entire emergency riser disconnection operation can be performed in as little as one hour.

To underscore the fact that such an operation is simple and proven technology, during the first 19 months of operation of the floating production facility in the Argyll Field in the North Sea, the production riser was pulled 17 times for weather. This also underscores possible weather sensitivity.

3.5.6 Operations Summary

In this section, some of the operational aspects of floating production systems in the southern Bering Sea are discussed.

The number and type of required support vessels will vary depending on specific requirements such as resupply intervals and distance to operations shore base. One workboat should be on location at all times for emergency evacuation and support purposes. Based upon limiting sea ice conditions, a number of ice-class workboats should be on site for the purpose of breaking up and diverting ice floes around the structure. These workboats will also be used as support vessels for offloading operations to shuttle tankers and possibly mooring operations for the storage tanker.

The advantage of producing wells with a semi-submersible from a subsea template is the ability to perform wireline workovers from the producing vessel. Also, since individual flowlines produce each well, workovers are possible without interrupting production from other wells. If satellite wells are incorporated into the production scheme, workovers of these wells must be performed by a separate drilling vessel.

Although the semi and storage tanker will be designed for the 100-year storm (omitting ice), the transfer of oil to the shuttle tankers is limited to substantially less severe sea states. Shuttle tanker mooring and hose connection to the storage tanker is limited to wave heights of about 10 to 15 feet. Product transfer will probably have to cease when waves build up to 15 to 20 feet although North Sea operations have at times exceeded these values. This limit will be somewhat controlled by relative tanker sizes. In addition, ice floe conditions may also contribute to some offloading and possibly production downtime, depending upon the amount of "buffer" storage available at the time and the duration of the ice condition.

An additional and very important operational constraint is the occurrence of a storm during the ice season. As was stated earlier, wave driven ice floes can produce enough impact force to damage vertical brace members. Given the susceptibility to this type of damage (converted semis), ice netting and the ability for quick disconnect and site abandonment is imperative.

3.5.7 Costs and Scheduling

Cost and scheduling information is included in Section 6.

3.5.8 Engineering Considerations

Recent technological developments in such areas as subsea production equipment and single point moorings have substantially increased the feasibility of floating production systems in even the harshest ocean environments. Floating production systems are particularly suited for early field production, marginal field production and early reservoir evaluation. The main advantages of floating production systems are substantial capital cost reduction compared to fixed structures, mobility for re-siting or site abandonment under severe conditions, and they are re-usable.

The field development of the southern Bering Sea lease sale areas evaluated in this study will require system designs to withstand and/or evade seasonal sea ice conditions. The technology exists for such design requirements although for some of the system components, such is beyond the state-of-practice. In particular, an offshore storage and offloading terminal has yet to be designed and constructed for such an environment. This is a critical area which will require future work. For example, the effect of superstructure icing on the SPM could disturb the function of exposed equipment such as hose connections or swivel joints. With respect to the actual SPM superstructure design, substantial model testing will be required to analyze and evaluate either the disconnection or submergence procedures.

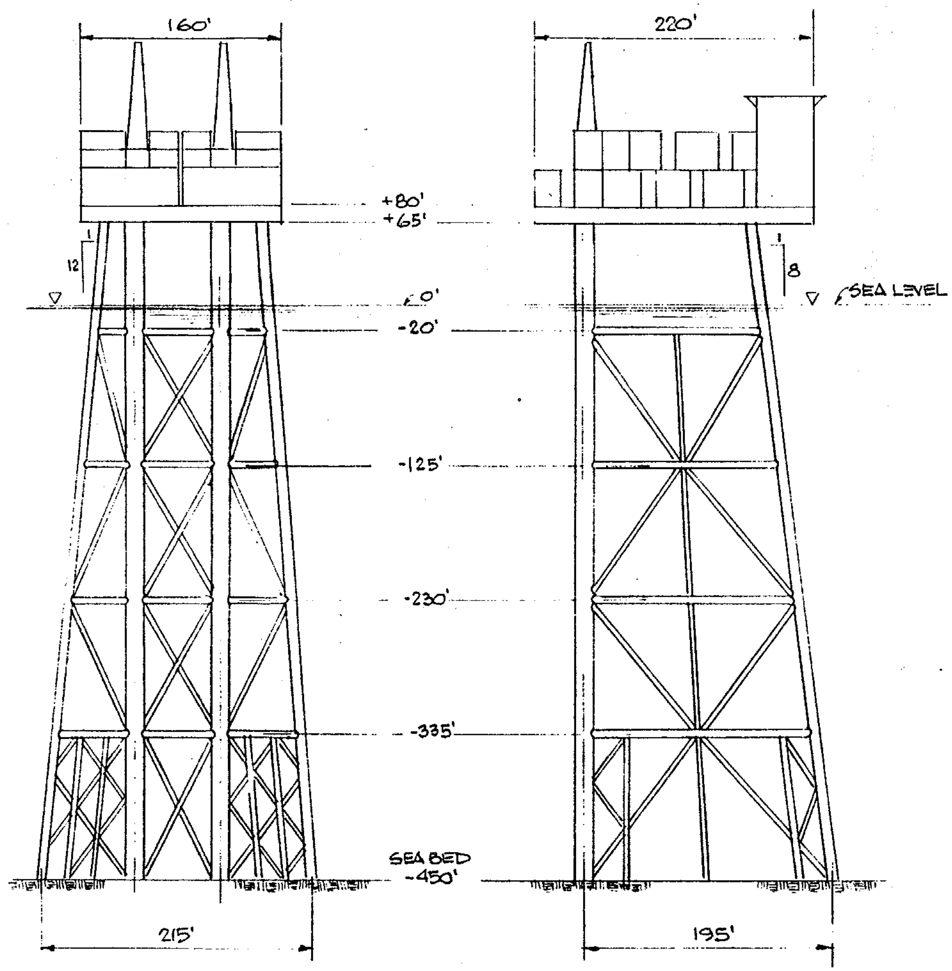
With regards to the production platforms, this study focused on the semi-submersible platform based upon their track record as effective production platforms and experience in drilling in similar ice environments similar to the Bering Sea. There are, however, other viable solutions as well, which warrant further study. An integrated floating production and storage system, such as that installed in the Castellon field offshore Spain by Shell Espana, is a potential alternative. In this system, a dedicated tanker moored by a rigid buoyant yoke performs the production and process operations as well as storage. Thus the system is fully self-contained. This may be a significant advantage in a sea ice environment in terms of quick site abandonment. As was stated earlier, however, a ship-shape vessel has yet to be designed and moored in a harsh environment such as the Bering Sea.

A significant advantage of a converted semi could be substantially reduced times for first production. The critical path for schedule shown in Section 6 is the construction of the subsea template and drilling of the wells. If only a small number of wells were pre-drilled (the schedule assumumes all six template wells pre-drilled), the platforms could be on-line much sooner. The advantage that the converted semi has over the purpose-built semi is substantially less design and in-yard construction time. The economics of such a decision will depend on such things as anticipated field production rate per well and scheduling of the other system components.

From an operational standpoint, more information is required relating structural ice strength to actual sea ice conditions. In particular, local loads due to wave-ice interaction need more study. In order to run preliminary economics, estimated downtimes are required which will be based upon such information. More accurate seasonal ice coverage data will also be required. Unique problems need also be identified which are associated with operating in or near ice areas and in very remote areas. Potential solutions to these problems need be identified.

3.5.9 Conclusions

Based on the work done herein, we would not recommend the use of semi-submersible floating production systems as the primary development concept for major fields due to unresolved questions about sensitivity to ice impact, evacuation strategy and higher maintenance sensitivity in a very remote area. We do believe that the systems discussed should be considered as companion systems to the primary fixed platform concepts.

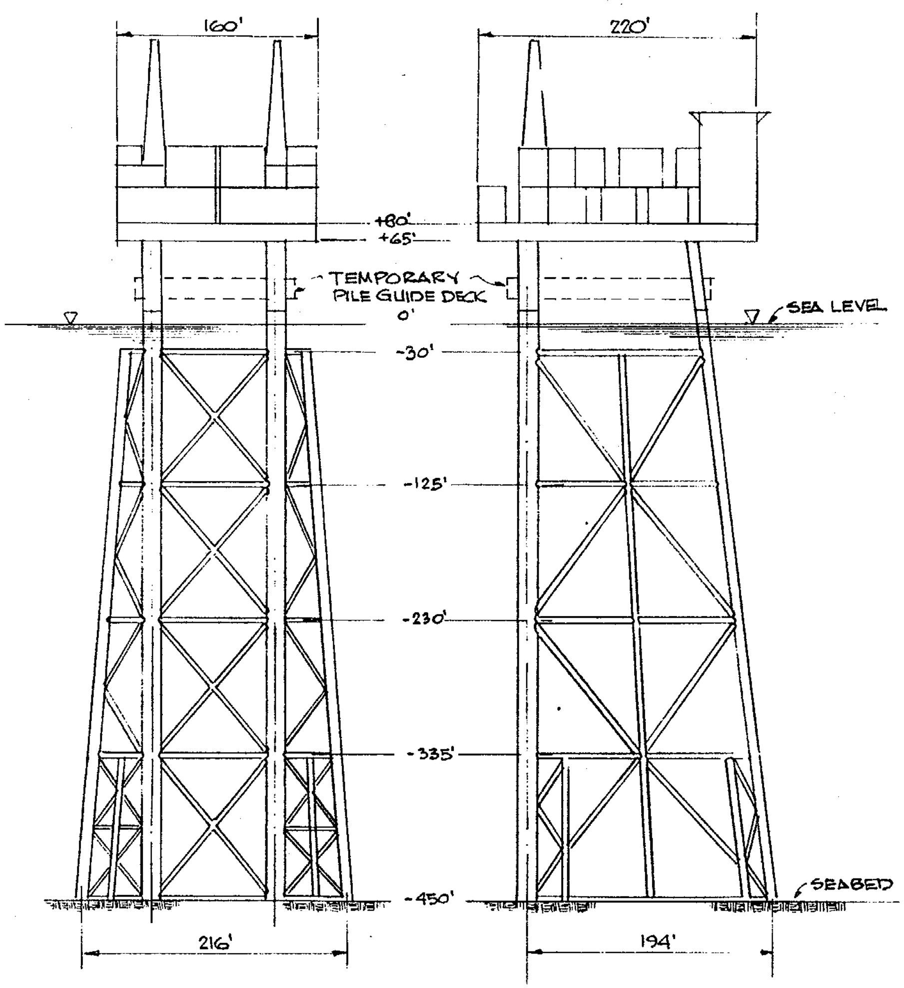


ADVANTAGES: GOOD DECK SUPPORT
3 EXTERIOR BATTERED FRAMING PLANES
CAN BE BARGE TRANSPORTED
CONVENTIONAL PILE DRIVING

DISADVANTAGES: LARGE ICE LOADS WITH MORE POTENTIAL FOR JAMMING

8 LEG TEMPLATE CONCEPT

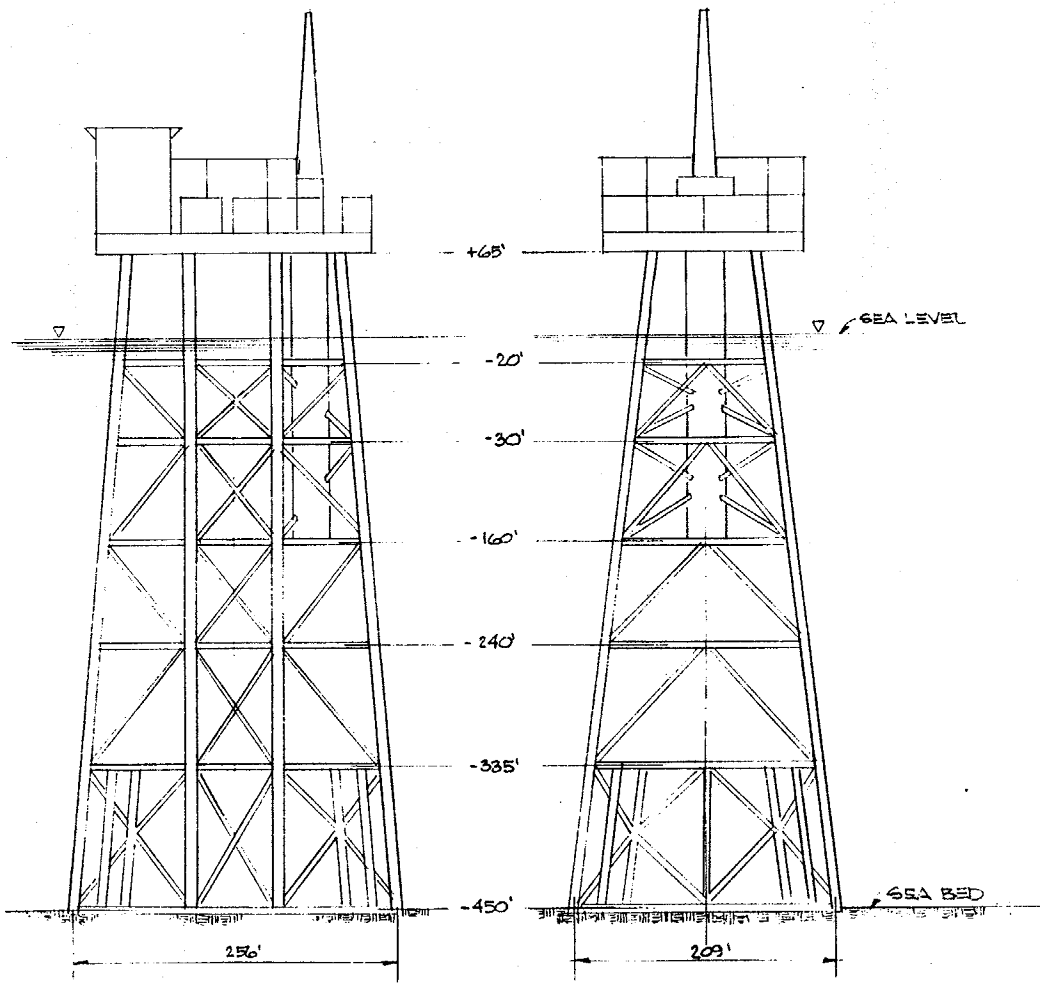
FIGURE 3-1



4 PLUS 4 TEMPLATE

FIGURE 3-2

ADVANTAGES:	MINIMIZES ICE FORCES 3 EXTERIOR BATTERED FRAMING PLANES CAN BE BARGE TRANSPORTED
DISADVANTAGES:	ONLY 4 DECK SUPPORT POINTS PILE DRIVING UTILIZES TEMPORARY DECK

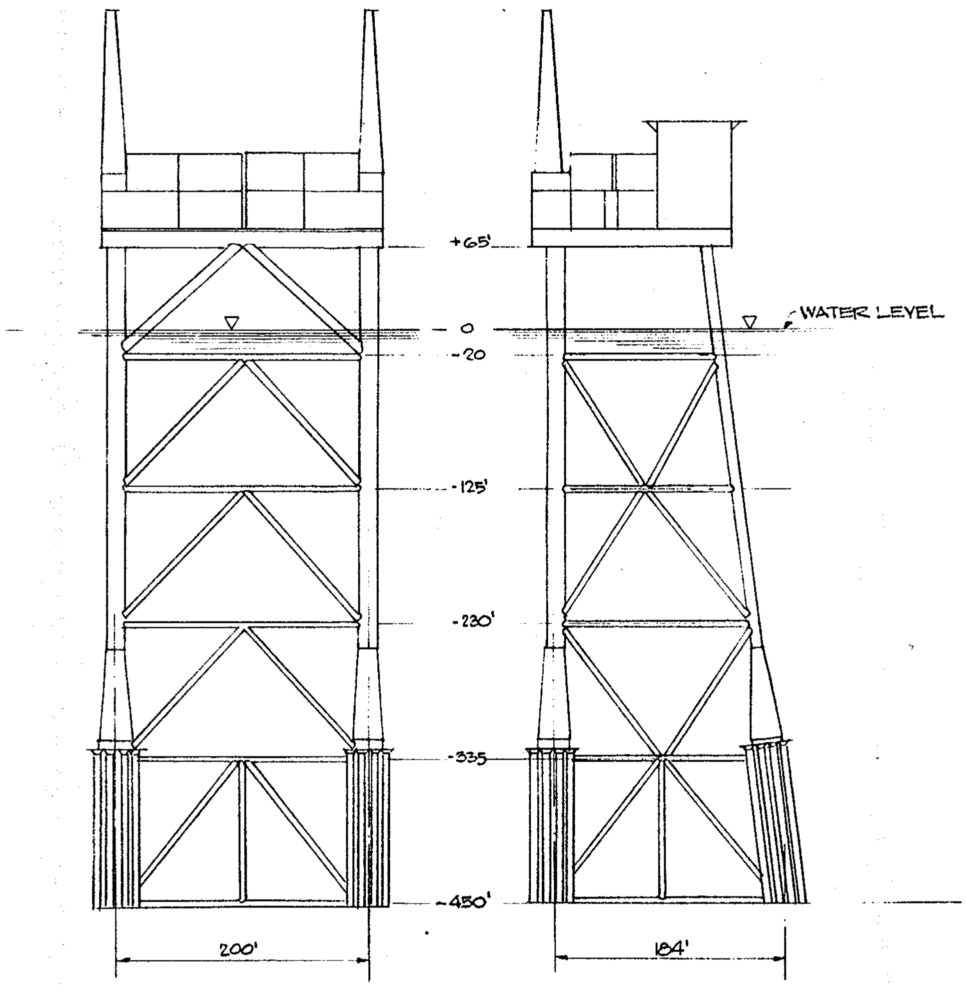


ADVANTAGES: RELATIVELY CONVENTIONAL DECK ARRANGEMENT

DISADVANTAGES: LOCAL CAISSON FRAMING LOADS
HIGH ICE & WAVE IMPACT LOADS
DECK STRUCTURE ADD-ON DIFFICULT DUE TO CAISSON SPLICE

8 LEG TEMPLATE WITH CAISSON

FIGURE 3-3



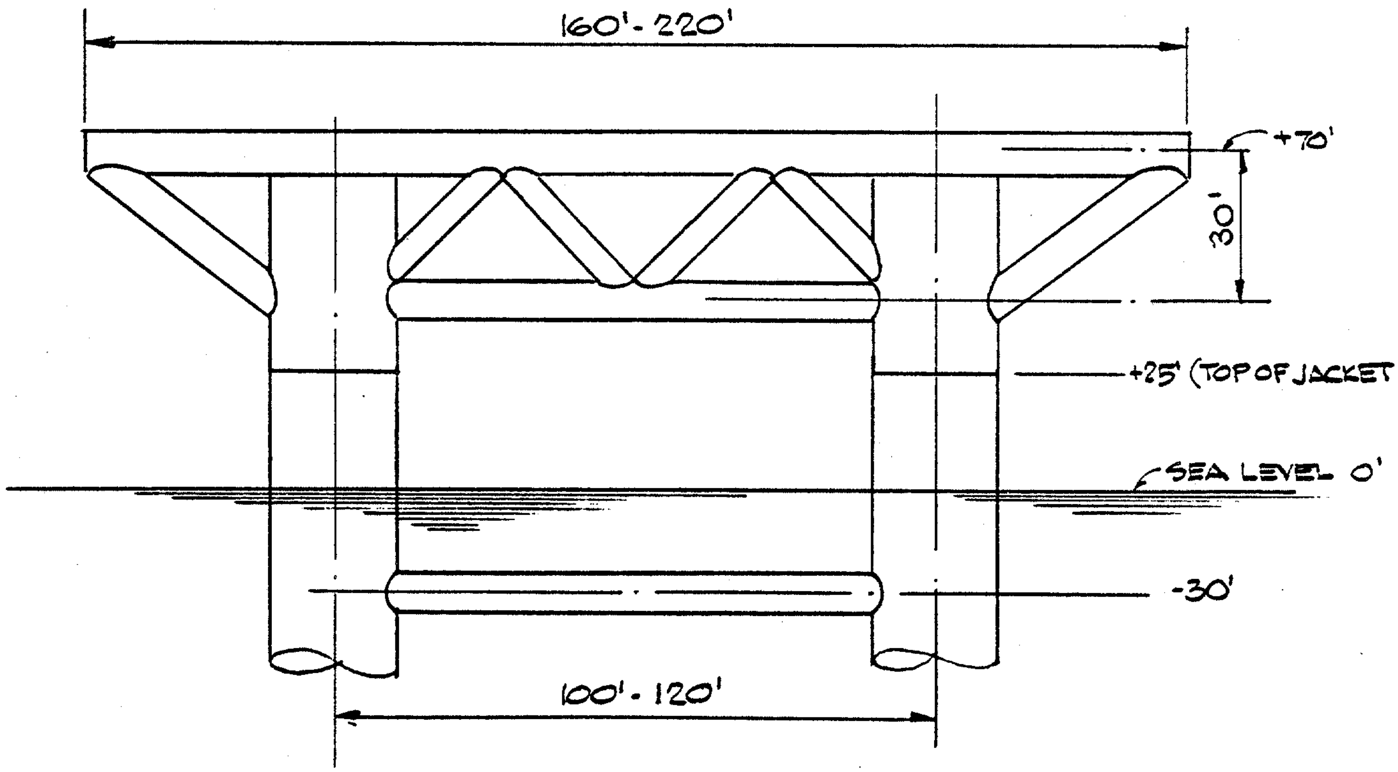
SELF FLOATING TOWER

FIGURE 3-4

ADVANTAGES: LARGE VERTICAL LEGS FOR WELL PROTECTION LEND THEMSELVES TO FLOATATION.

DISADVANTAGES: TRANSPORTATION STRESSES/RISKS
LESS FLEXIBILITY IN FRAMING ARRANGEMENT
- WELL LEG REQUIREMENT
- FLOATATION REQUIREMENT

DECK ARRANGEMENT NOT OPTIMUM FROM SAFETY STANDPOINT.

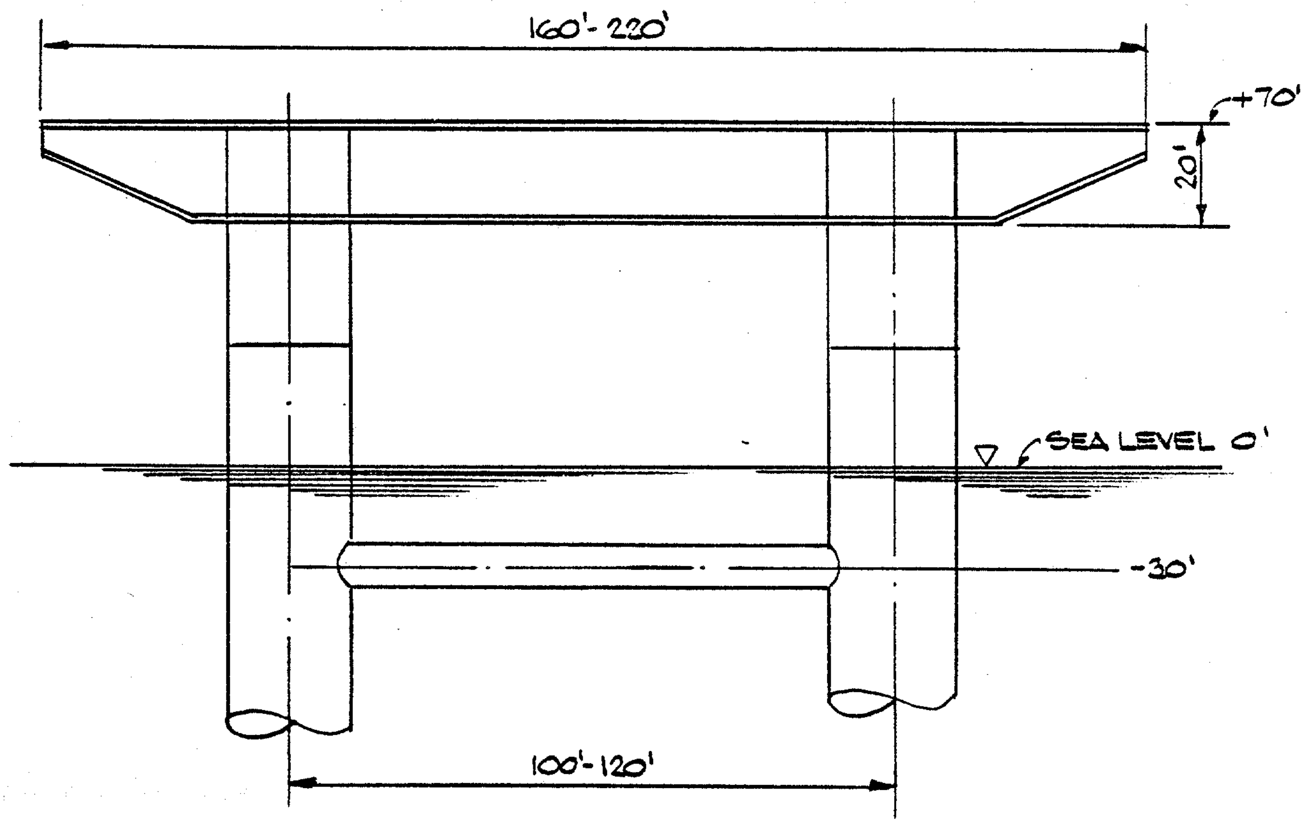


DEEP TRUSS

ADVANTAGES: MINIMIZES WEIGHT
MINIMIZES WAVE FORCE

DISADVANTAGES: POSSIBLE ICE IMPACT
ON LOWER CHORDS
HIGH LOCAL STRESSES
ON LARGE LEGS.
MODULE REACTION
POINT SENSITIVITY

FIGURE 3-5

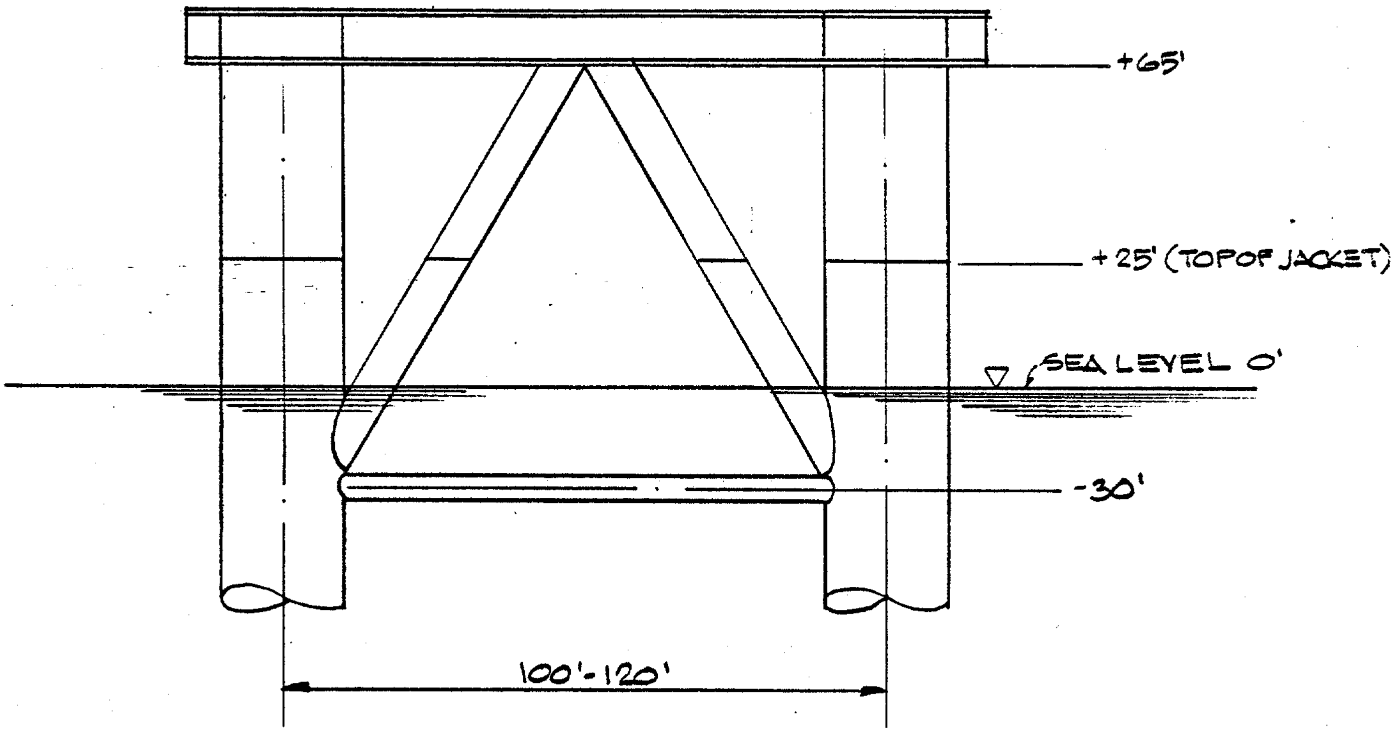


DEEP BOX GIRDER

ADVANTAGE: LIQUID STORAGE POTENTIAL
INSENSITIVE TO MODULE REACTION POINTS

DISADVANTAGE: HEAVY
HIGH WAVE SLAM FORCE

FIGURE 3-6

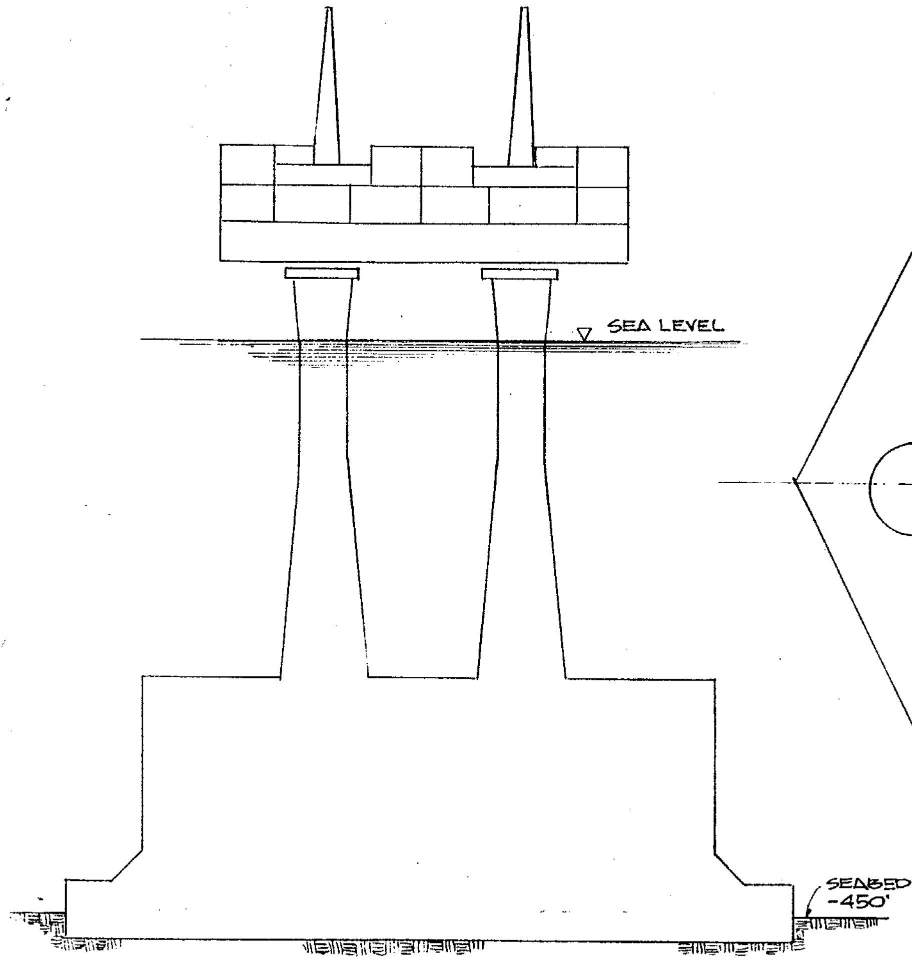


SHALLOW GIRDER WITH K BRACE
SHALLOW TRUSS WITH K BRACE

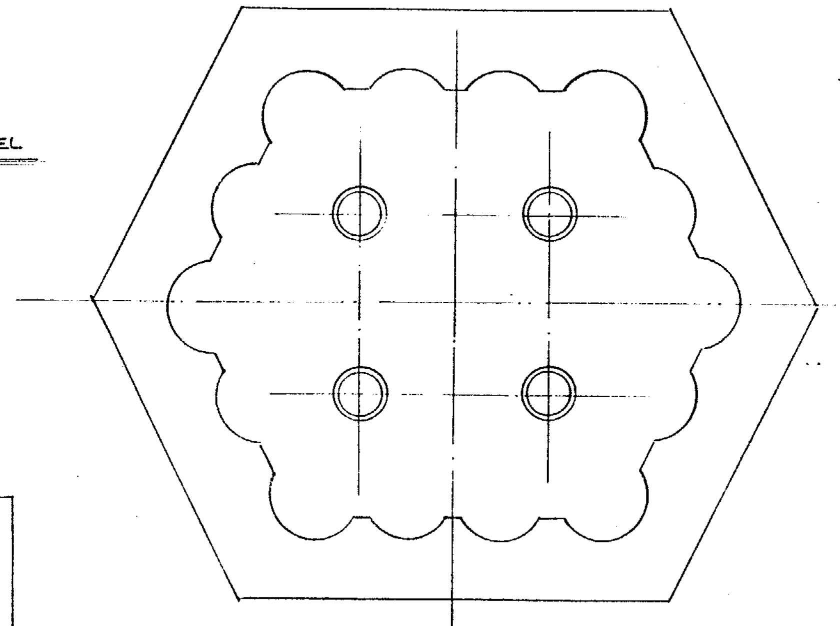
ADVANTAGES: BRACED FRAME/LIGHTER
LESS MIDSPAN DEFLECTION

DISADVANTAGES: LOCAL ICE SLAM LOADS ON BRACES
MORE POTENTIAL FOR ICE JAMMING
DECK STRUCTURE INSTALLATION COMPLICATED.

FIGURE 3-7



ELEVATION



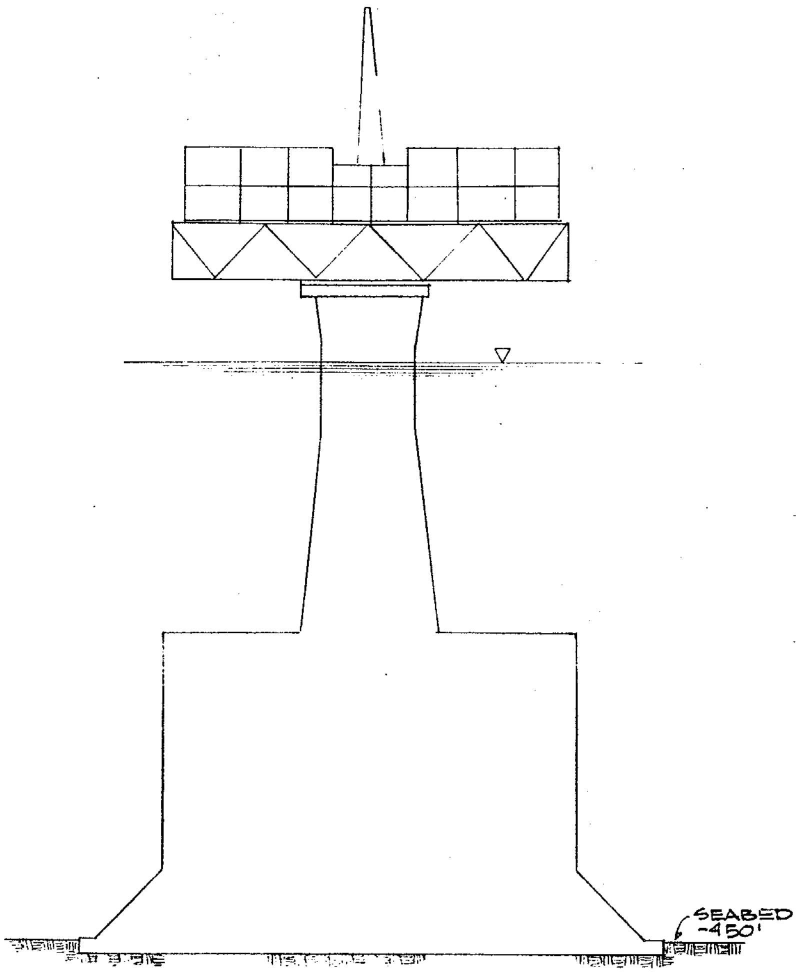
SECTION @ WATERLINE

ADVANTAGES:
GOOD STORAGE POTENTIAL
GOOD TOPSIDE SUPPORT
GOOD STRUCTURE REDUNDANCY
GOOD STABILITY AFLOAT

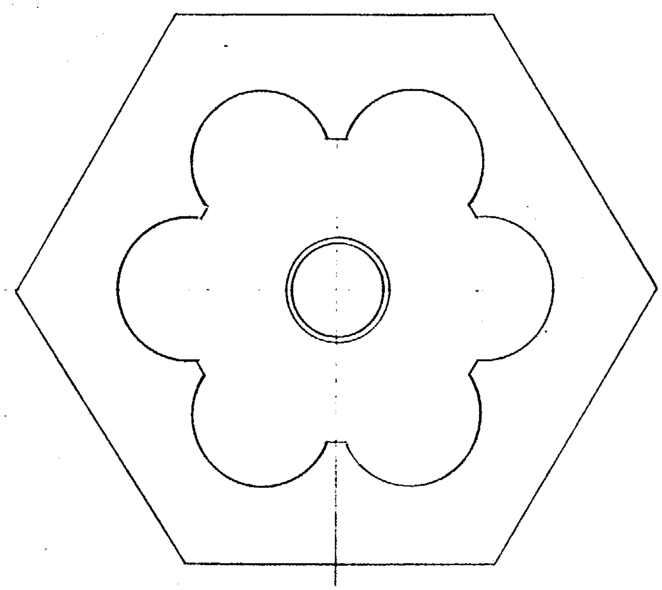
DISADVANTAGES:
POSSIBLE ICE JAMMING

CONDEEP CONCEPT
4-LEGGED DRILLING & PRODUCTION

FIGURE 3-8



ELEVATION



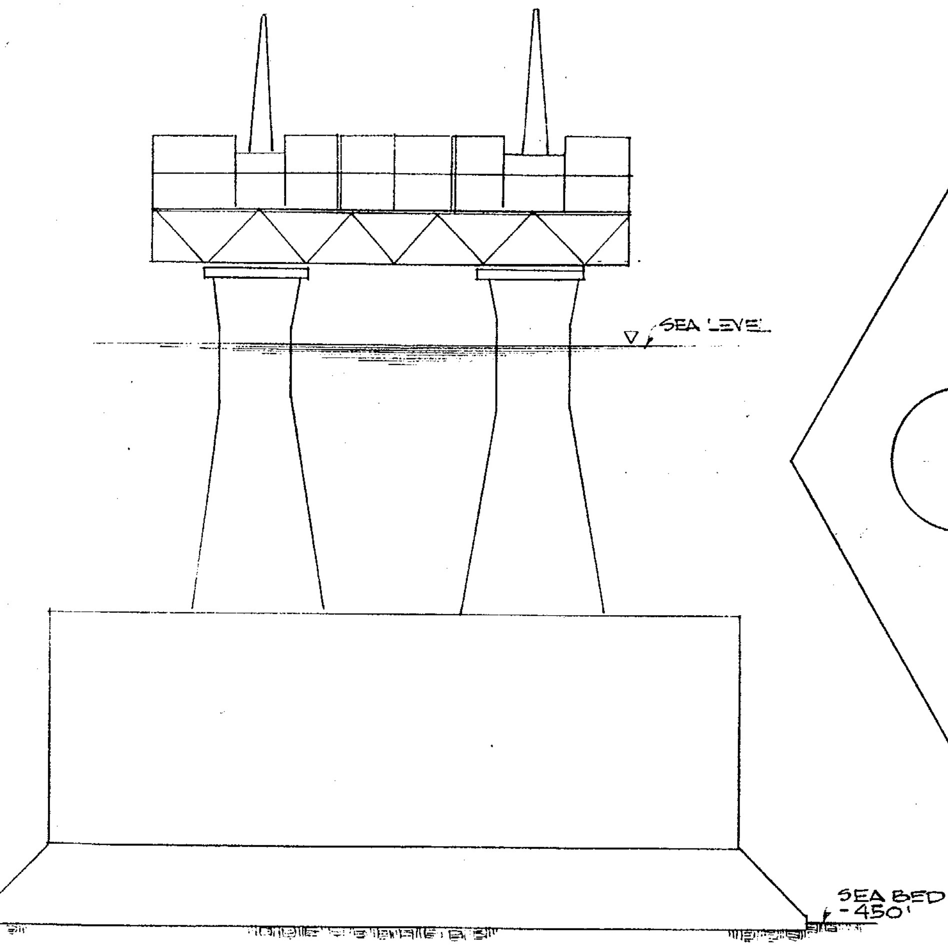
SECTION @ WATERLINE

ADVANTAGES: GOOD ICE PERFORMANCE
GOOD STORAGE POTENTIAL

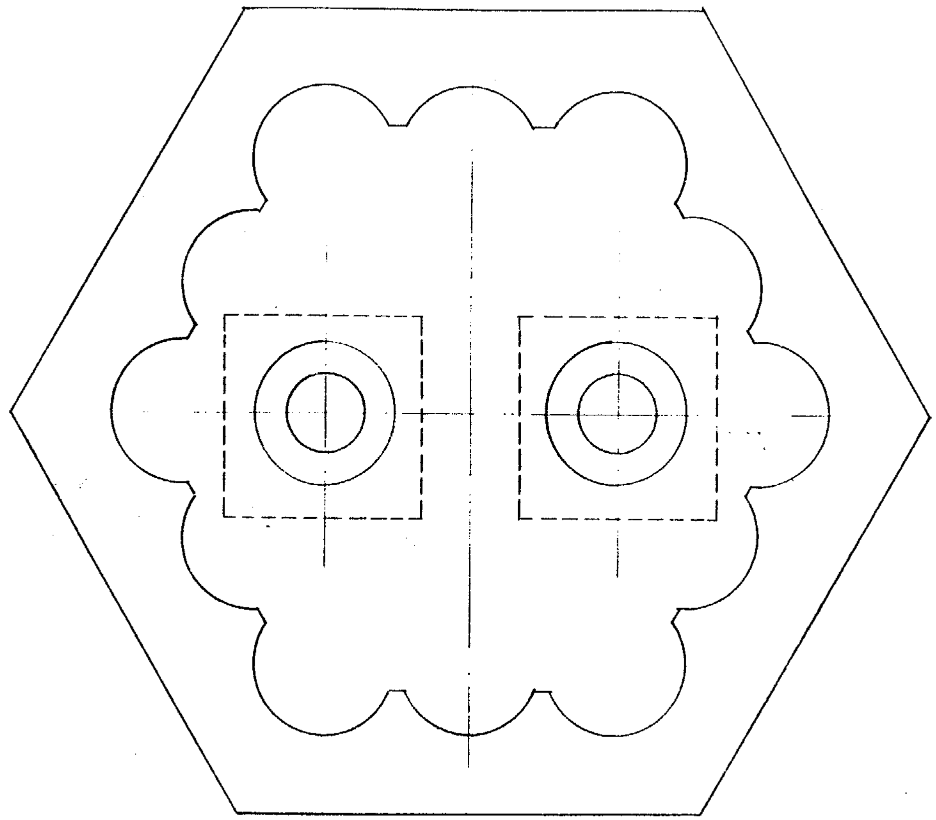
DISADVANTAGES: POOR STRUCTURE REDUNDANCY
TOPSIDE LAYOUT RESTRAINT
FAIR STABILITY WHILE AFLOAT

CONDEEP CONCEPT
SINGLE LEG DRILLING & PRODUCTION

FIGURE 3-9



ELEVATION



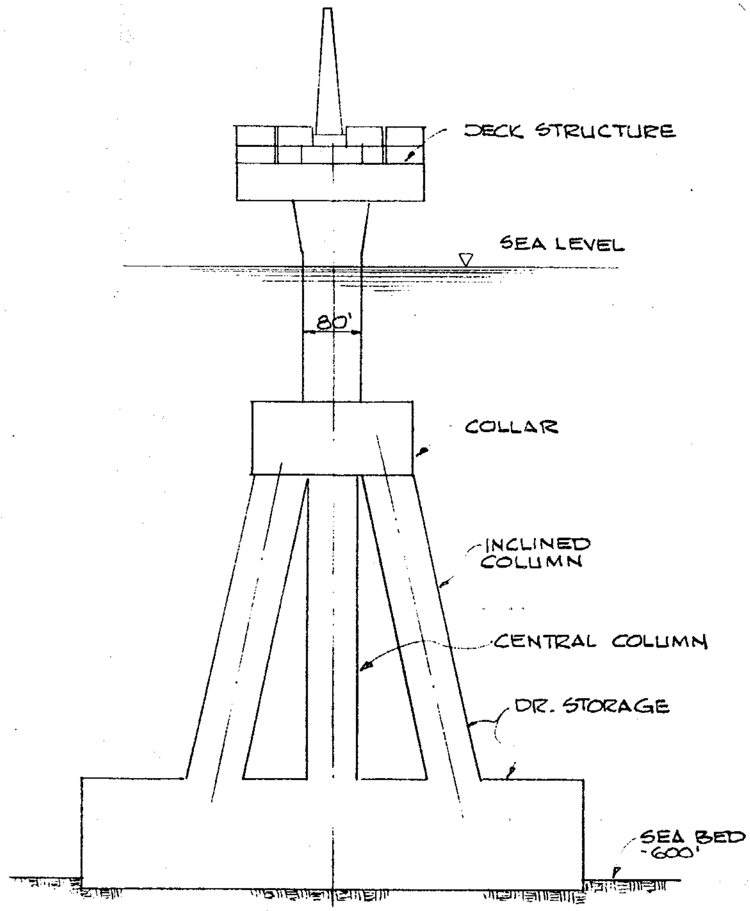
SECTION @ WATERLINE

TWIN TOWER DRILLING & PRODUCTION

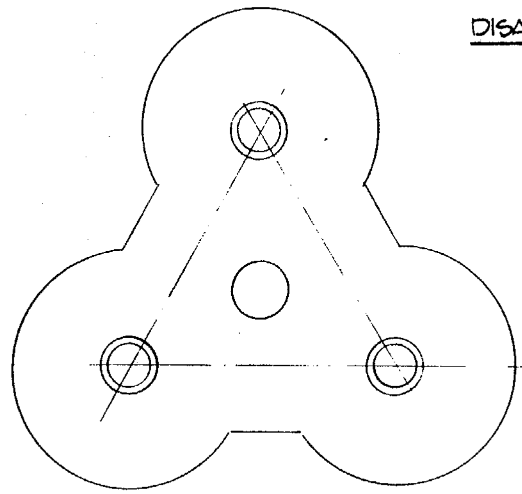
FIGURE 3-10

ADVANTAGES: GOOD STRUCTURE REDUNDANCY
GOOD STORAGE CAPACITY

DISADVANTAGES: UNPROVEN STABILITY AFLOAT
POSSIBLE ICE JAMMING



ELEVATION



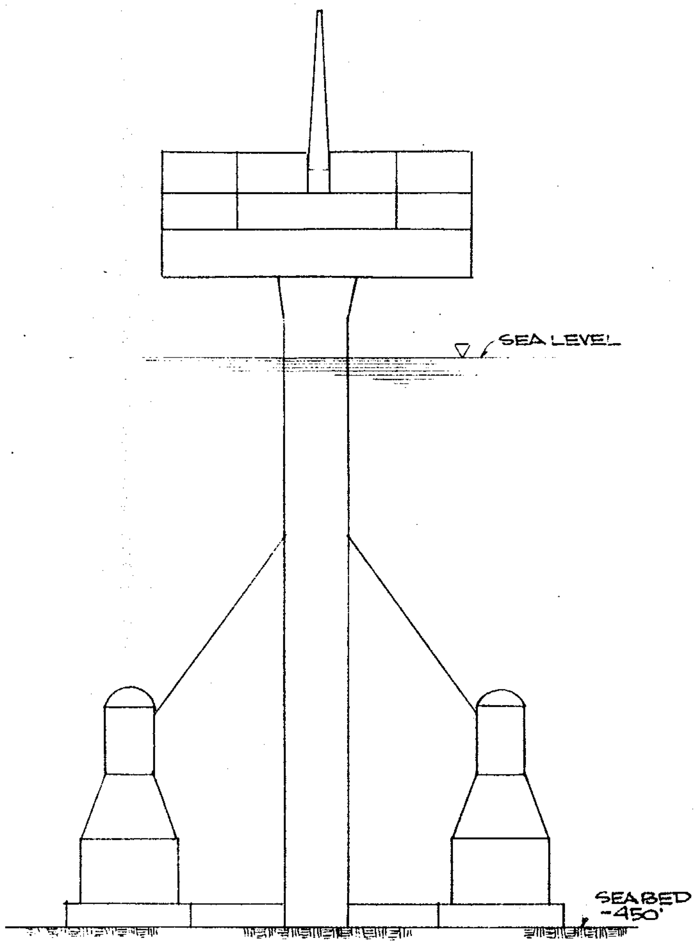
SECTION @ BASE

ADVANTAGES: EFFICIENT CONFIGURATION FOR USE IN DEEP WATER
GOOD ICE PERFORMANCE

DISADVANTAGES: UNPROVEN CONSTRUCTION METHODS

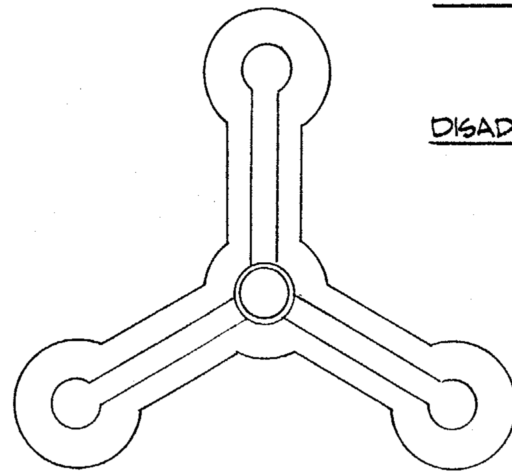
TRIPOD W/ MONOTOWER

FIGURE 3-11



ELEVATION

CHEVRON CONCEPT

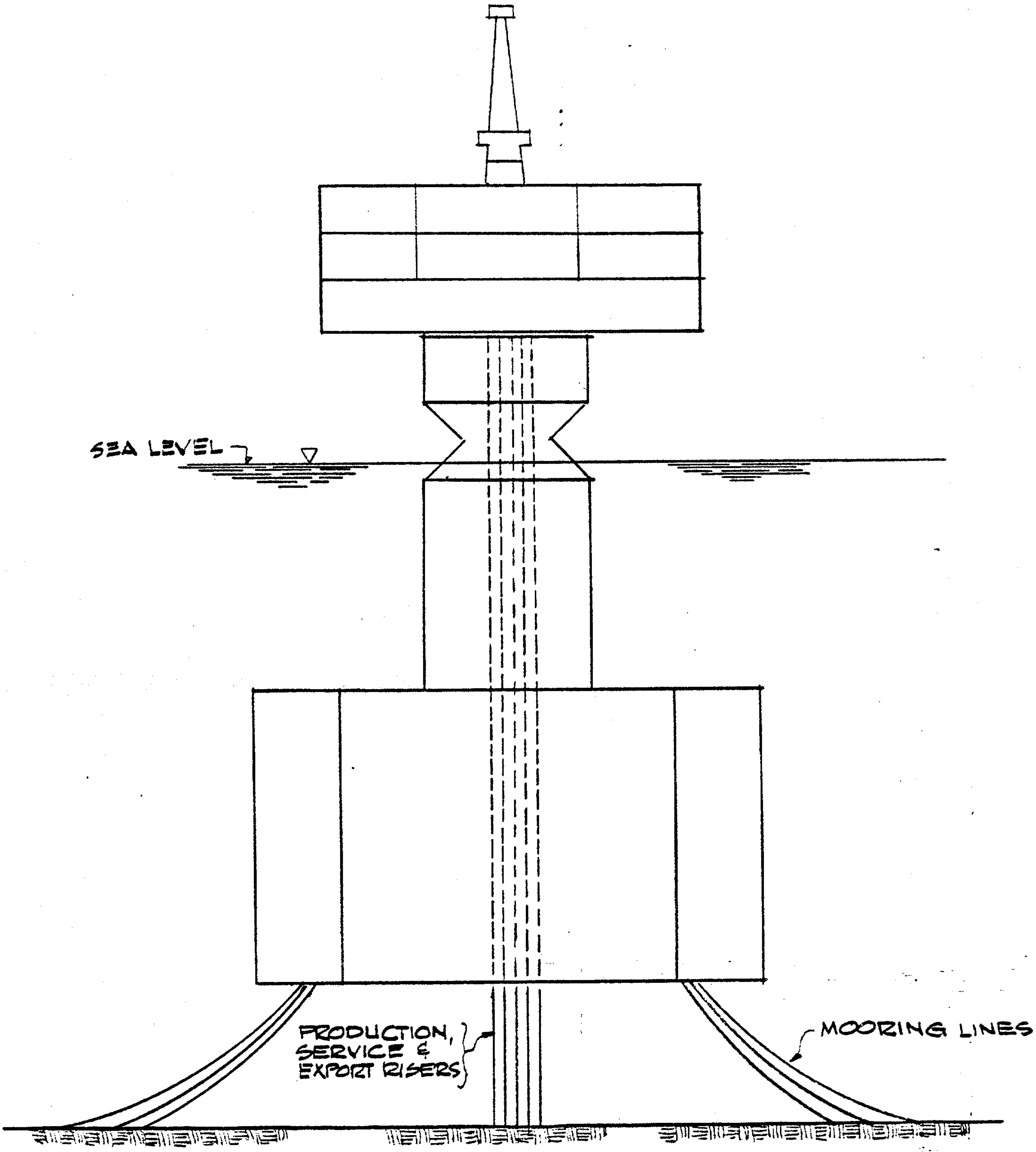


SECTION @ WATERLINE

ADVANTAGES: EFFICIENT CONFIGURATION
FOR USE IN DEEP WATER
GOOD ICE PERFORMANCE
POTENTIAL CONCRETE SAVINGS

DISADVANTAGES: UNPROVEN CONSTRUCTION METHODS
LOW TORSION STRENGTH
POOR SLIDING RESISTANCE

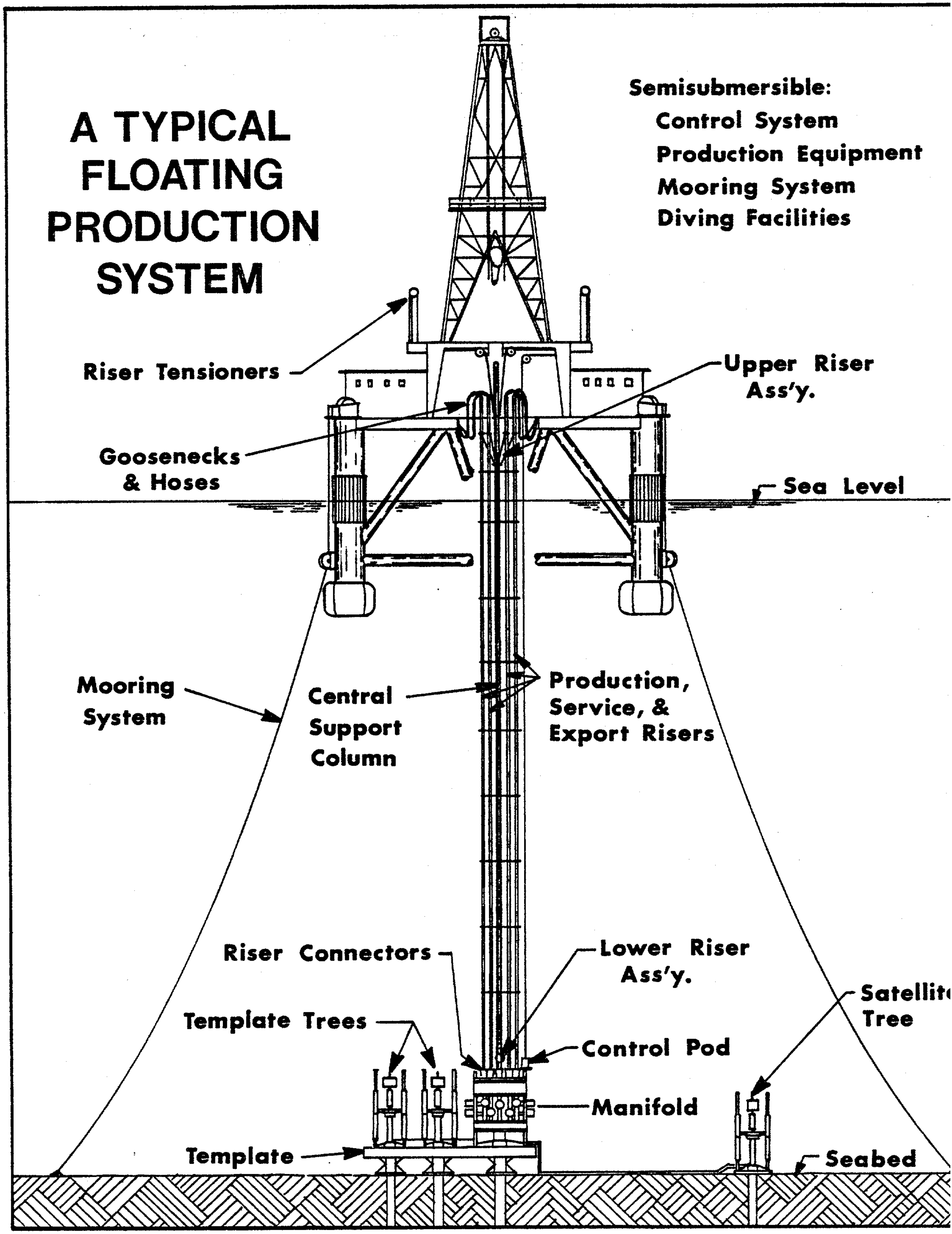
FIGURE 3-12



ELEVATION

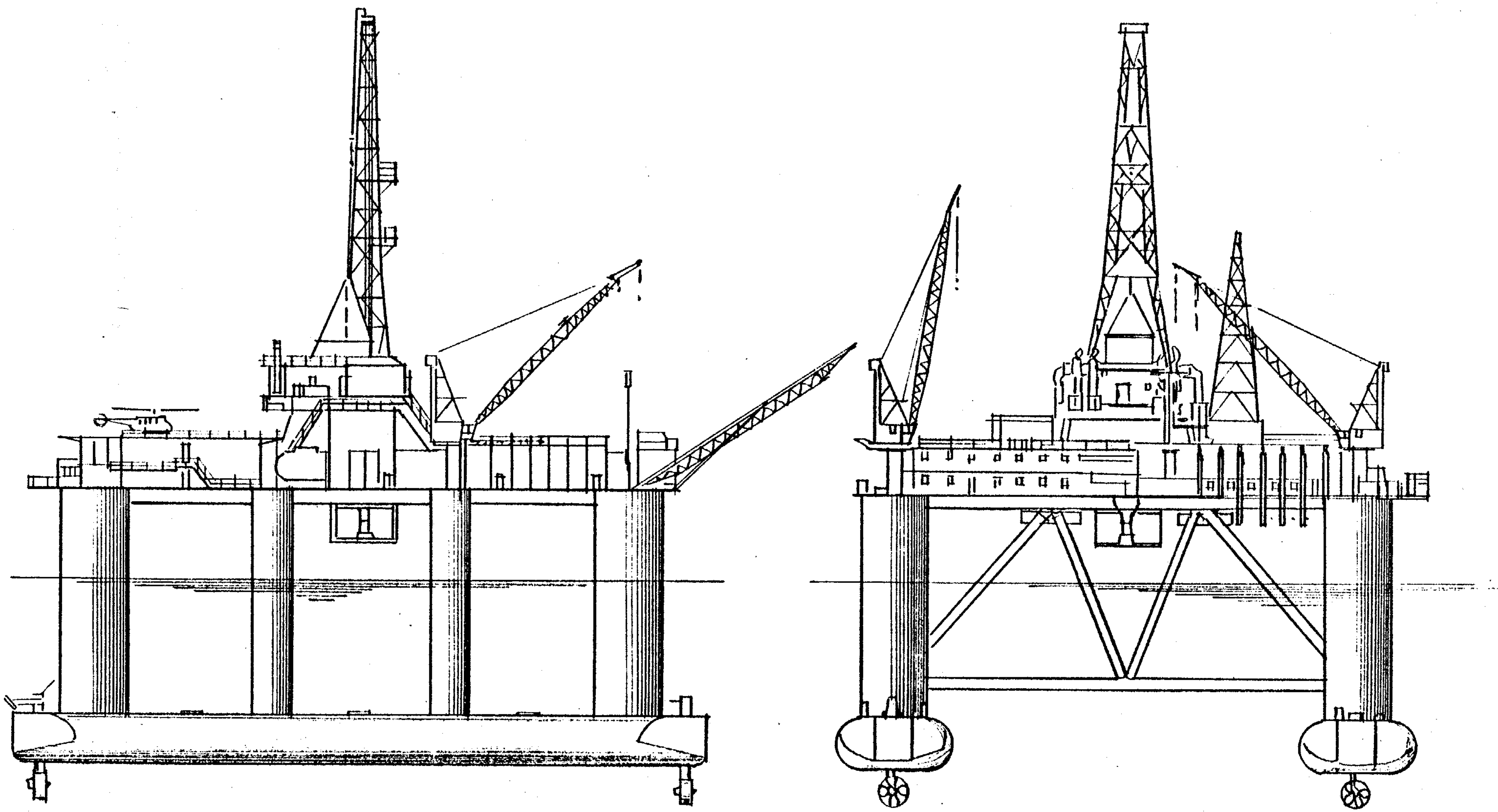
ARCTIC FLOATING PRODUCTION CAISSON CONCEPT

FIGURE 3-13



A TYPICAL FLOATING PRODUCTION SYSTEM
Semisubmersible:
Control System
Production Equipment
Mooring System
Diving Facilities
Riser Tensioners
Upper Riser Ass'y.
Goosenecks & Hoses
Sea Level
Mooring System
Central Support Column
Production, Service, & Export Risers
Riser Connectors
Lower Riser Ass'y.
Template Trees
Control Pod
Manifold
Template
Seabed

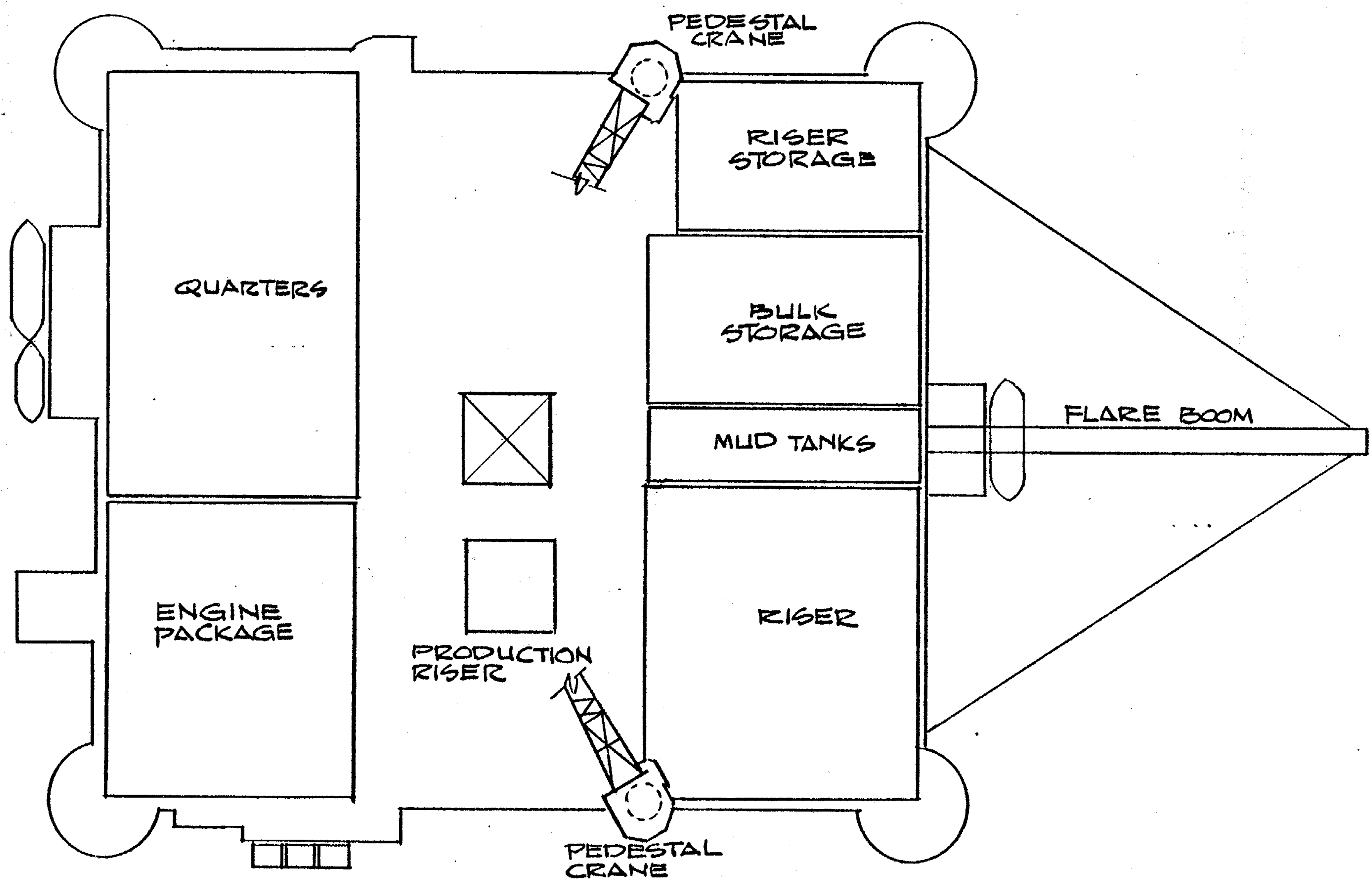
Figure 3-14



SEDCO 700 SERIES - CONVERTED PRODUCTION SEMI

GENERAL PROFILES

FIGURE 3-15

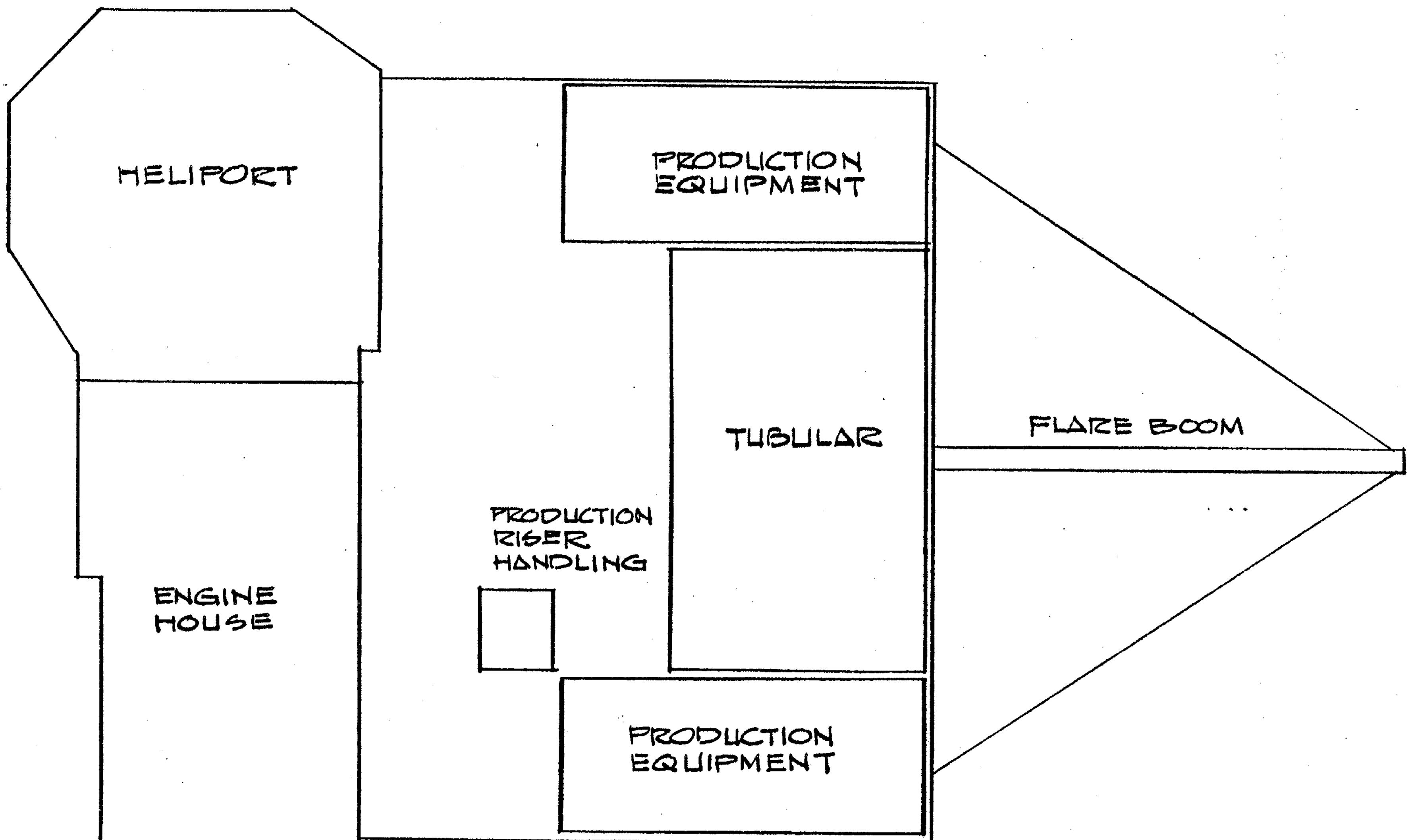


PLAN - MAIN DECK

SEDCO 700 SERIES - CONVERTED PRODUCTION SEMI

GENERAL ARRANGEMENT

FIGURE 3-16

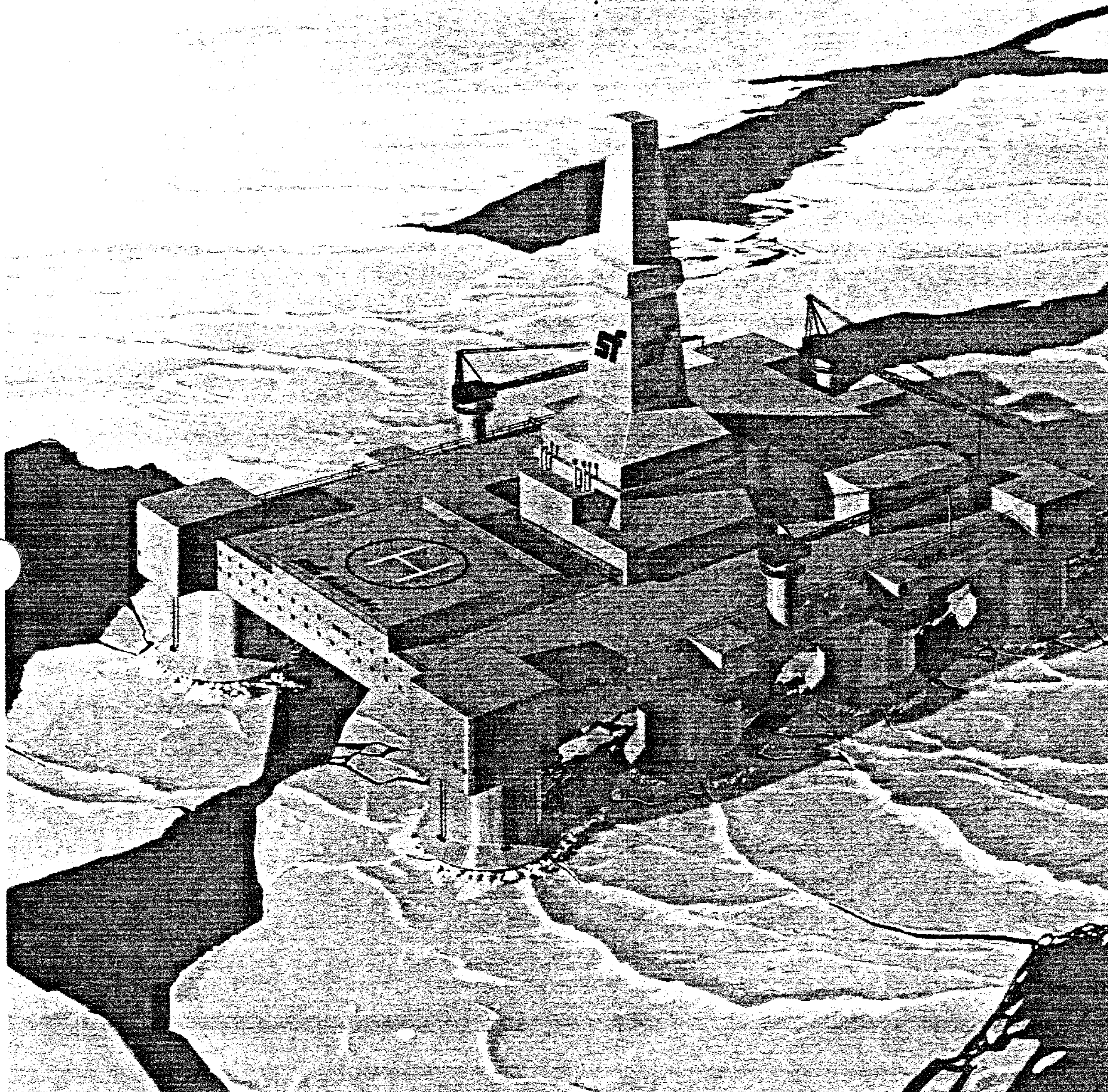


PLAN-ROOF DECKS

SEDCO 700 SERIES - CONVERTED PRODUCTION SEMI

GENERAL ARRANGEMENT

FIGURE 3-17



SaFe SUBARCTIC SEMI-SUBMERSIBLE

Figure 3-18

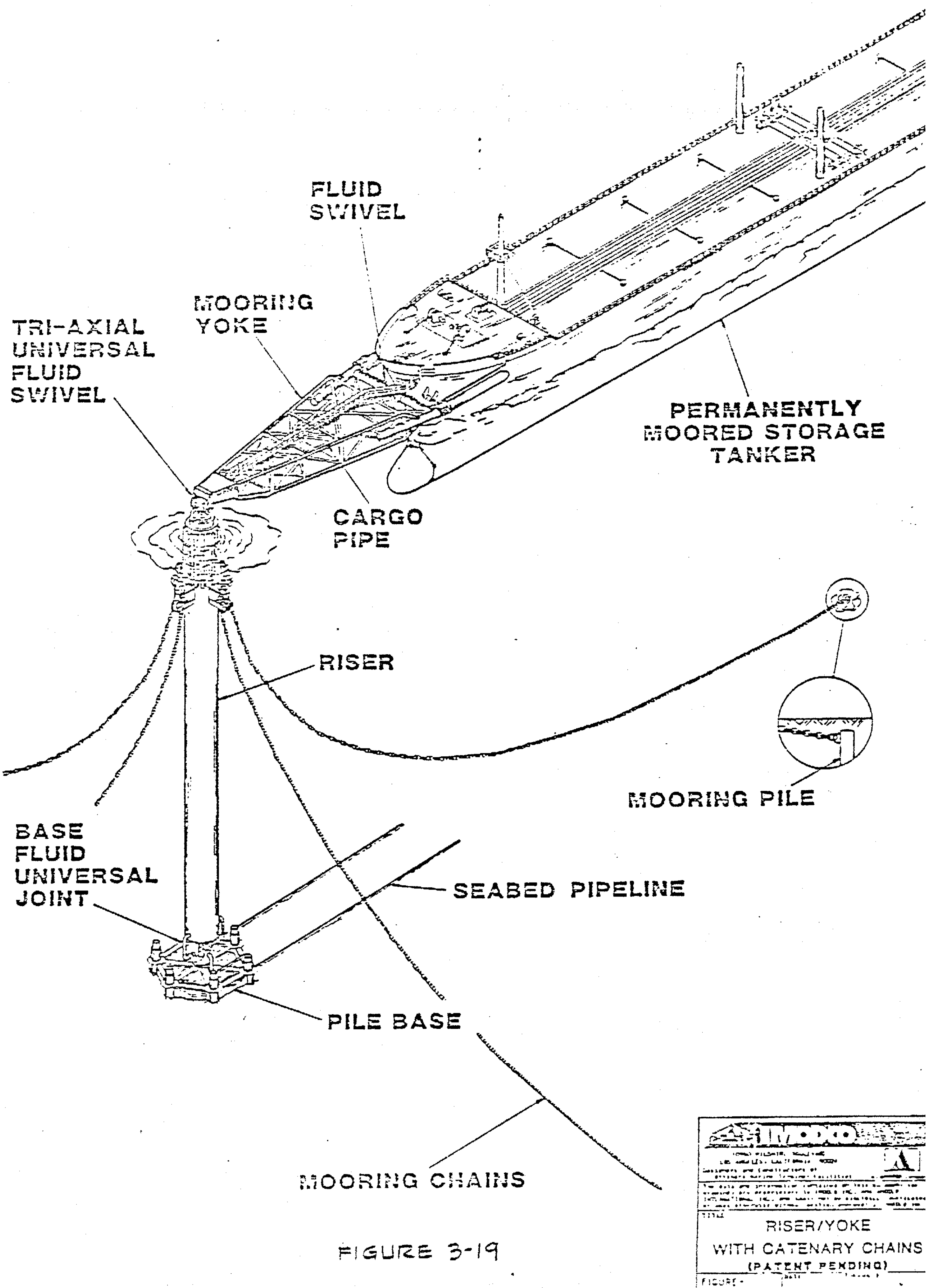

FLUID SWIVEL
MOORING YOKE
TRI-AXIAL UNIVERSAL FLUID SWIVEL
PERMANENTLY MOORED STORAGE TANKER
CARGO PIPE
RISER
MOORING PILE
BASE FLUID UNIVERSAL JOINT
SEABED PIPELINE
PILE BASE
MOORING CHAINS
RISER/YOKE
WITH CATENARY CHAINS
(PATENT PENDING)

FIGURE 3-19

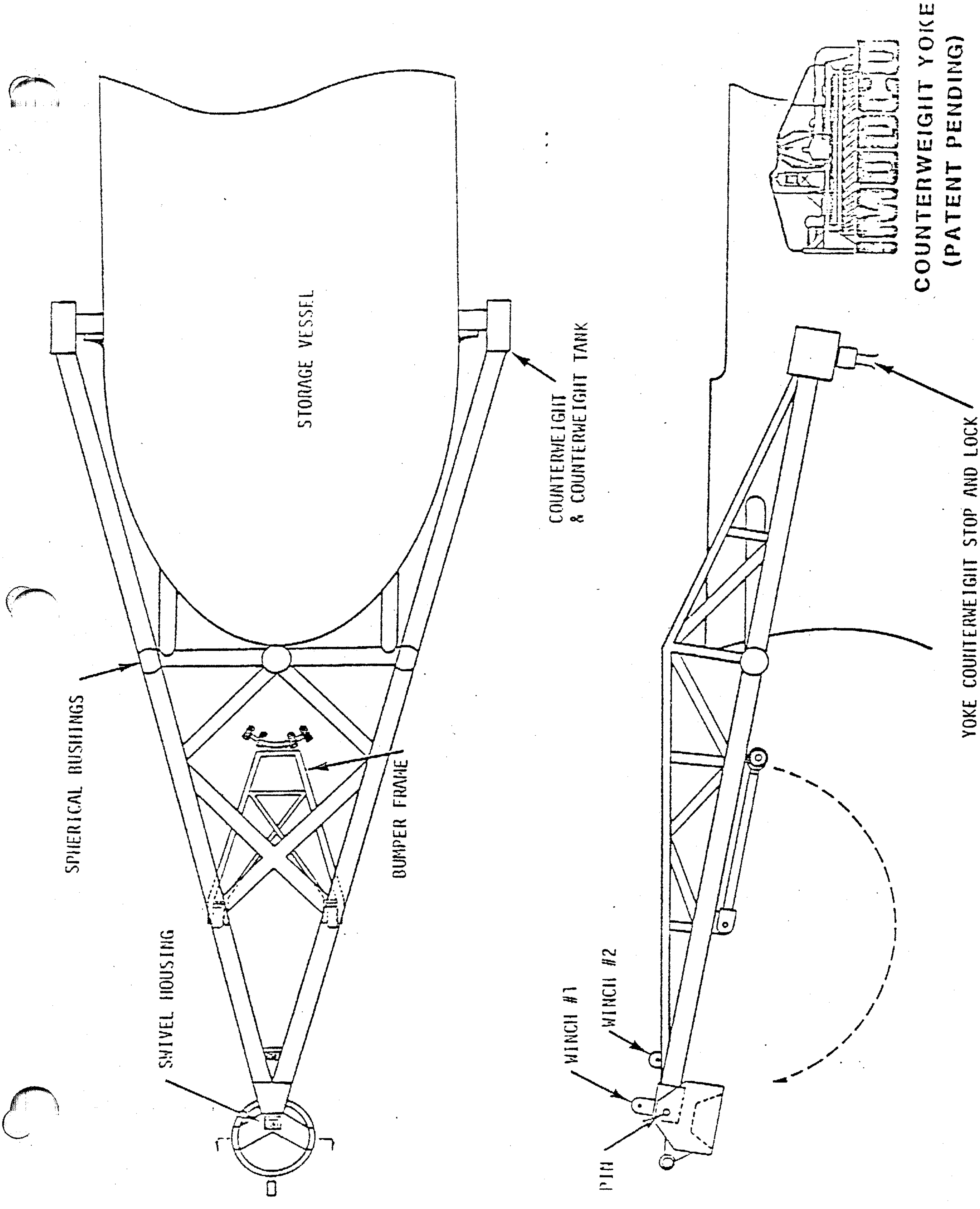
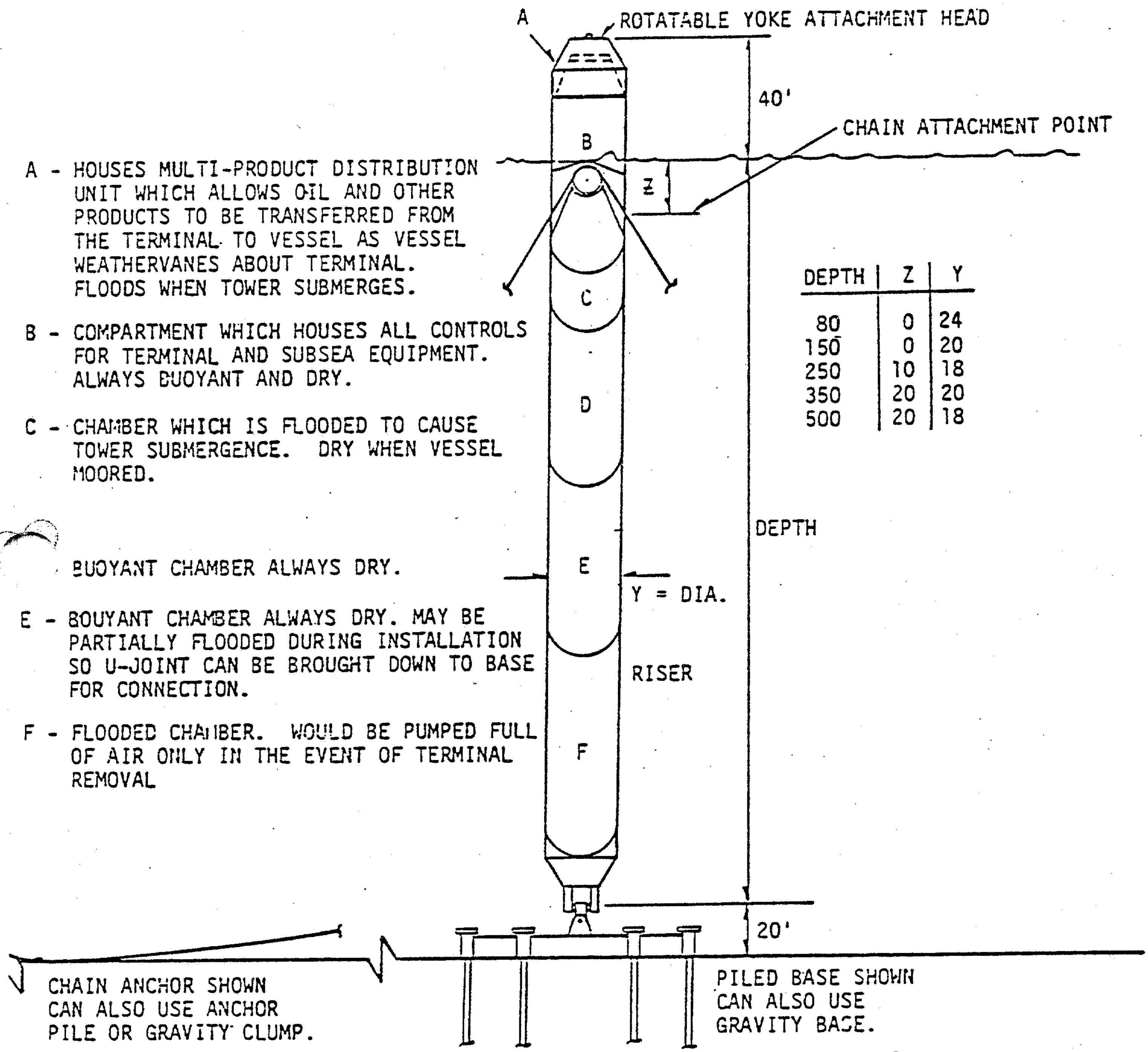

STORAGE VESSEL
COUNTERWEIGHT
& COUNTERWEIGHT TANK
SPHERICAL BUSHINGS
BUMPER FRAME
SWIVEL HOUSING
COUNTERWEIGHT YOKE
(PATENT PENDING)
YOKE COUNTERWEIGHT STOP AND LOCK
WINCH #1
WINCH #2
PIN

FIGURE 3-20



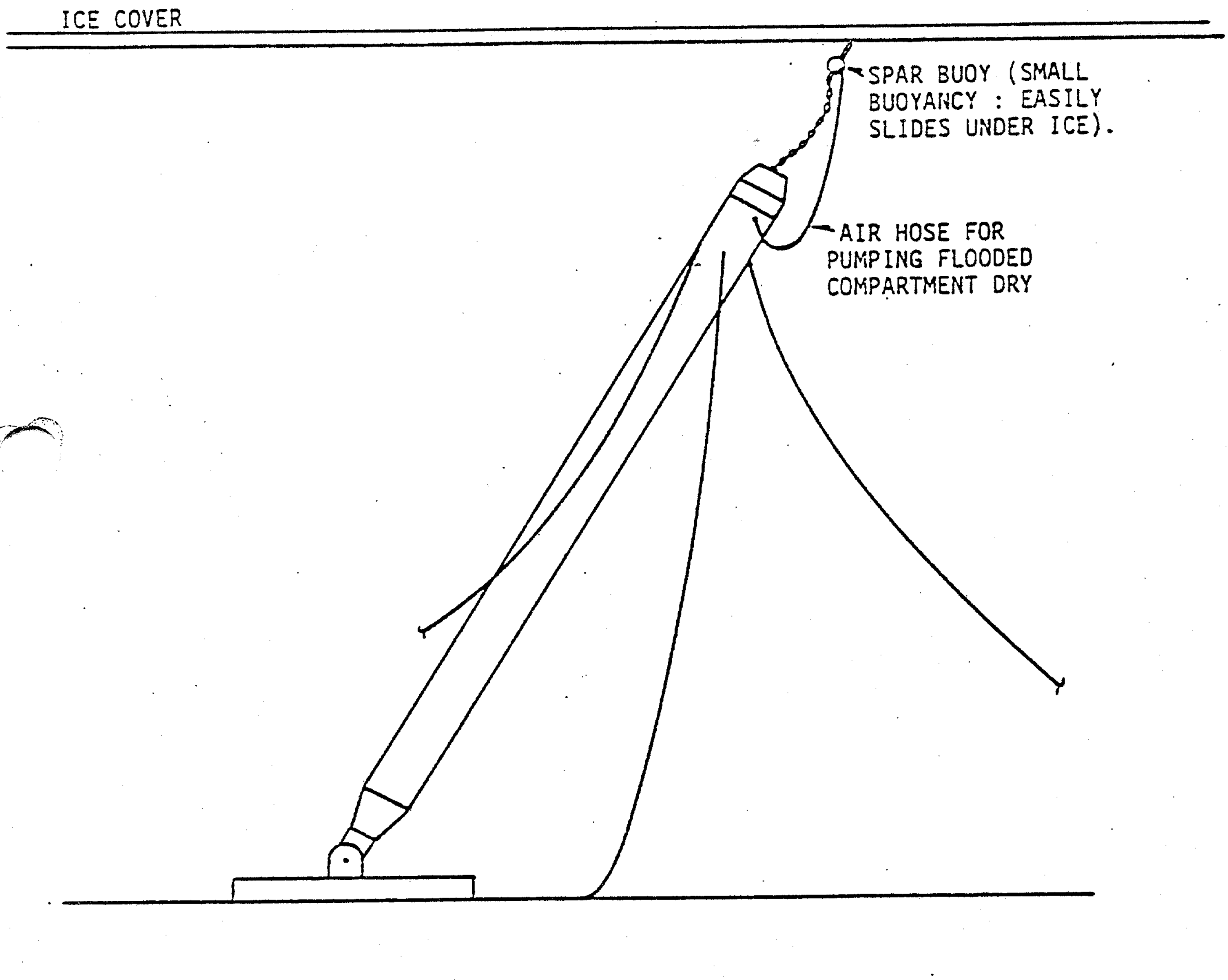
DEPTH	Z	Y
80	0	24
150	0	20
250	10	18
350	20	20
500	20	18

FIGURE 3-21

IMODCO

STORAGE TANKER MOORING TERMINAL

STORAGE TANKER MOORING TERMINAL PURPOSELY SUBMERGED FOR ICE SEASON WITH FLOODED CHAMBER



IMODCO

FIGURE 3-22

TABLE 3-1
CONCRETE STRUCTURES IN THE NORTH SEA

Type	Name	Operator	Year	Site Data: Water Depth ft	Site Data: Foundation Soil	Base: Slab ft2	Base: Skirts ft	Storage Capacity MMBBLS	Concrete Volume yd3
Doris	Ekofisk	Phillips	1973	230	Dense fine sand	Flat A = 80,000	1.3 concrete ribs	1.0	118,000
Condeep	Beryl A	Mobil	1975	394	Dense find sand over clay	Conical Domes A = 67,000	10 steel 1.6 concrete	0.93	72,000
Condeep	Brent B	Shell	1975	460	Stiff clay with sand layers	Conical Domes A = 67,000	13 steel 1.6 concrete	1.0	85,000
Doris	Frigg CDP1	Elf	1976	322	Dense fine sand	Flat, ring-shaped A = 60,000	None	---	78,000
Sea Tank	Frigg TP1	Elf	1976	341	Dense find sand over clay	Flat A = 60,000	6.5 concrete	---	92,000
Doris	Frigg MCPO1	Total	1976	308	Dense fine sand	Flat ring-shaped A = 60,000	None		
Condeep	Brent D	Shell	1976	460	Stiff clay with sand layers	Conical Domes A = 84,000	14.8 steel 1.6 concrete	1.0	85,000
Condeep	Statfjord A	Mobil	1977	476	Stiff clay	Conical Domes A = 84,000	10 steel 1.6 concrete	1.3	115,000
Andoc	Dunlin	Shell	1977	500	Stiff clay with sand layers	Flat A = 114,000	13 steel	0.85	116,000
Condeep	Frig TCP2	Elf	1977	335	Dense find sand over clay	Conical Domes A = 100,000	11.5 steel 1.6 concrete	---	65,000
Doris	Ninian	Chevron	1978	446	Stiff clay with sand layers	Flat A = 166,000	11.5 steel	---	186,000
Sea Tank	Brent C	Shell	1978	450	Stiff clay with sand layers	Flat A = 111,000	10 concrete	0.65	137,000
Sea Tank	Cormorant	Shell	1978	492	Stiff clay with sand layers	Flat A = 104,000	10 concrete	1.0	150,000

TABLE OF CONTENTS

SECTION 4.0

LIST OF DRAWINGS AND FIGURES

SECTION 4.0

LIST OF TABLES

SECTION 4.0

4.0 STEEL PILE SUPPORTED STRUCTURES

4.1 Introduction

4.1.1 Scope of Work

The preceding chapter describes how the 4 plus 4 template was qualitatively evaluated to be the steel platform concept most generally promising for the Southern Bering Sea. The 4 plus 4 template has been carried through to preliminary designs for conditions typical of three basins in the Bering Sea: St. George Basin, Navarin Basin, and the North Aleutian Shelf. The purpose of this work is to confirm the technical feasibility of the structure, and to develop designs sufficiently to enable preparation of cost estimates suitable for use in deciding bidding strategies. In this regard, it is not requisite to complete the designs in detail. However, it is important to assess their sensitivity to variations in conditions such as production rate, water depth, environmental loads, and soil profile.

4.1.2 Organization of Section

First, the following text introduces the structural configuration that was studied (Section 4.1.3) and the premises and assumptions used to develop the preliminary designs (Section 4.2). Then sections 4.3, 4.4, and 4.5 present the designs for the St. George, Navarin, and North Aleutian Basins, respectively. Each of these latter sections describes the specific operational and environmental conditions, structural analysis, resulting preliminary design, material estimation and construction planning for a base case design. Additionally, each base case design's sensitivity to plausible ranges in key operational and environmental variables is discussed. The assumed base case production rate is 300,000 barrels per day for all basins; base case water depth is 450 feet for the Navarin and St. George Basins, and 300 feet for the North Aleutian Shelf. Section 4.6 concludes the chapter with a critique of the designs and general conclusions.

4.1.3 Structure Configuration

Drawing 101 shows the proposed structural configuration. Key elements and features of the base case 4 plus 4 template are:

- Two large diameter vertical legs (A2 and A3) through which the well conductors can be run.

- Two parallel battered back legs (B2 and B3) serving as runners for loadout and launch at sea. These legs are large diameter at the upper two bays of the structure, but make a transition to more conventional smaller diameters below.

- Two battered corner legs (A1 and A4) and two double-battered corner legs (B1 and B4) which run from roughly elevation -30 feet downwards; they complete the template and spread the base of the structure.

- The absence of horizontal and vertical bracing from elevations +65 to -30 feet. This minimizes the potential for ice jamming and limits direct ice loadings to only the large diameter, armored legs.

- Large box girders which support the deck modules and complete the moment-resisting frame at the top of the structure.

- A steel pile foundation typically comprising six main piles and twelve skirt piles clustered three around each corner leg. It is assumed that the conductor drive pipe can functionally replace main piles for vertical loading at the drill legs.

The legs piercing the water surface are spaced a minimum of 100 feet center-to-center. This is assumed to be the minimum spacing to provide ice clearing in the St. George and North Aleutian areas, such that ice floes will not arch between legs to produce loading across the entire cross-section of the structure. Larger spacings are objectionable along rows A and B because they tend to spread the template excessively and do not allow sufficient deck cantilevers to help balance the bending moment at the tops of the legs.

Virtually the same platform configuration has been studied for each of the three basins. Base dimensions and the number of horizontal levels vary with water depth. Base widths are 170 feet for 300 feet of water and 183 feet for 450 feet of water. Even for the upper bound water depth of 600 feet, the base width of 195 feet is within the construction capacity and seaway limits of known potential fabricators. Using the same configuration at each site does not result in consistently optimized designs, particularly not for the foundations. However, this approach provides a reasonably accurate basis for estimating costs and is consistent with the intended level of detail for this project.

4.2 Premises and Assumptions

4.2.1 Materials

The large payloads and cold environment contemplated for these structures demand a strong tough steel. Additionally, it is desirable to limit member wall thicknesses in view of the difficulty in maintaining strength and weldability for greater thicknesses, and high yield steels help in this regard. Proposed steels and their applications are as follows:

- ASTM A-537 Grade 1 (Normalized) in the underwater zone.
- ASTM A-537 Grade 2 (Quenched and Tempered) or ASTM A-633 Grades C or D (Normalized) in the ice and air zone and in legs A2, 3, B2, B3 down to elevation -125'.
- Common yield strength steels, can be used for secondary braces and stiffeners, mudmats and other noncritical elements or elements loaded only during installation.

With the exception of the last item, these steels are classified as Group II, Type A steels by API RP 2A, and have a yield strength of 50 ksi. They should be specified with supplemental requirements for Charpy V-notch toughness of at least 25 foot-pounds at +32°F for the underwater zone and 25 foot-pounds at -40° F for the ice and air zone. Additionally, vacuum degassing may be specified for steel which will be used in joint cans and brace stubs.

4.2.2 Codes

The basic guideline for designing the platforms is the American Petroleum Institute's "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," 12th edition (API RP 2A). The weight of internal stiffeners in the large diameter legs has been estimated from past designs for hydrostatic collapse based on the Structural Stability Research Council's "Guide to Design Criteria for Metal Structures."

4.2.3 Structure Guidelines and Assumptions

For normal operating conditions, nominal stresses are limited to the basic allowable values given in API RP 2A. A one-third increase in these basic allowable values is permitted for storm or maximum ice conditions; a seventy percent increase is permitted for strength requirement seismic loads.

Several geometric limits have been observed in selecting brace sizes. Primary brace sizes were generally selected to achieve slenderness ratios of 100 or less and diameter/wall thickness ratios of 48 or less. The former avoids column buckling at small stresses and the latter avoids premature local buckling before development of a member's full plastic load and moment capacity.

Diameter/wall thickness ratios for unstiffened legs (A1, A4, B1 and B4) do not meet the API RP 2A requirement of 60 or less, thus, they should be investigated for local buckling during fabrication, tow and installation. However, it is assumed that local buckling is precluded for in-place conditions because piles will be grouted into these legs.

All legs are assumed flooded but braces are assumed buoyant.

Joint cans and brace stubs have not been designed in detail. Evaluation of punching shear and fatigue is beyond the scope of this project. However, a twenty percent allowance in weight has been made for thickening at the joints, miscellaneous stiffeners, secondary braces, mill and weld tolerance.

4.2.4 Foundation Assumptions

The piles are designed to achieve safety factors (the ratio of ultimate pile capacity to applied load) of at least 2.0 for normal operating conditions and 1.5 for design storm, ice, or earthquake conditions. Pile capacities have been estimated from the given undrained shear strength (Figure 4-1) and internal friction angle (Figure 4-2) profiles using the provisions in Section 2.6.4 of API RP 2A. Figure 4-3 shows pile capacity curves for sand and clay. Compressive capacity is based on both skin friction and tip bearing; in the case of sands, the tip bearing is limited by the skin friction of the internal soil plug. For tensile loads, the tip suction is assumed to be zero. Note that the crust associated with the clay profile has been neglected in determining capacities, on the assumption that the crust may be absent in local pockets.

Mudline stiffness coefficients of the soil-pile system have been estimated from past experience and rough calculations. It is especially important to consider the foundation stiffness when computing forces due to ground shaking, because these forces are a function of the structure's dynamic characteristics. For a 7.5-foot diameter pile, the following mudline stiffness coefficients are assumed:

	Axial (kip/inch)	Lateral (kip/inch)	
Cohesive			
Soft	5000	90	(neglects crust)
Medium	5800	575	(includes crust)
Stiff	7600	1100	(includes crust)
Cohesionless			
Loose	11000	300	
Medium	13000	600	
Dense	14000	1000	

The soil-pile bending stiffness and coupling between bending and translation tend to be secondary influences on overall platform response; they are assumed to be those of a pile fixed at roughly 30 feet below the mudline.

4.2.5 Loads

The following in-place load cases have been considered explicitly:

D = Dead weight of the jacket, appurtenances, piling, grout, deck support structure, deck module steel and dry deck equipment.

L = Maximum live load, such as drilling consumables, hook load, and fluids.

B = Jacket buoyancy.

H = Hydrodynamic load due to design wave acting statically.

W = Wind load

C = Current load

E = Strength requirement earthquake

I = Ice loads--the worst of either a consolidated rafted ice floe failing in crushing or a warm compression ridge failing in double shear. For the Navarin Basin, it is assumed that the ice can bridge between legs, so that the entire cross-section of the structure piercing the water surface is loaded. At St. George Basin and the North Aleutian Shelf, ice is assumed to load only the cross-sections of the legs piercing the water surface. This is discussed in Section 1.

S = Local ice pressure due to wave-driven chunks of ice slamming into the platform legs. It is assumed that no more than two legs can be simultaneously loaded.

Transportation, installation, fatigue and ductility analyses are beyond the scope of this study, but the material estimations are made with some contingency for these effects.

Hydrodynamic forces have been computed by the Morison-O'Brien equation using $C_D = 0.7$ and $C_M = 2.0$. Six inches are added to all member diameters to account for marine growth and appurtenances or secondary framing not explicitly included in the analytical models. Wave kinematics have been computed by Airy (linear) theory and combined with current kinematics prior to computing hydrodynamic forces. Designs waves are applied statically.

Wind loads are computed by equation 2.3.2-2 from API RP 2A:

$$F = 0.00256\ (V)^2\ C_s\ A$$

where

V = wind velocity in miles per hour

C_s = shape coefficient = 1.0

A = projected area of platform

Ice loads are formulated in Section 1. They are applied at elevation 0 feet.

The following load case combinations have been studied:

1. D + L + B (operating, no lateral)
2. D + L + B + H + W + C (storm)
3. D + 0.75L + B + E (strength seismic)
4. D + L + B + I + W (ice floe or ridge)
5. D + L + B + S + W (ice slam)

Note that light payload cases have not been included; consideration of such is unnecessary because the very heavy deck loads preclude uplift at the base of the jacket. However, live loads have been reduced 25 percent when combined with earthquakes, as suggested by API RP 2A. Design wave, wind plus current or ice plus wind are assumed to act colinearly along either lateral axis or the diagonal of the platform. The design earthquake acceleration spectrum is applied along either lateral axis of the structure with the same spectrum scaled by 2/3 and 1/2 applied simultaneously along the orthogonal lateral and vertical axes, respectively.

Consistent with standard practice, design environmental loads are applied independently; that is, the design storm, earthquake and ice are assumed to occur at different times. Ice loads are superposed with design wind, but no hydrodynamic load is added because the presence of ice cover is assumed to preclude formation of significant waves. It is possible that some combinations of loading, such as ice plus seismic, could produce high loadings. Investigation of these conditions was not done, and due to the dynamic character of the ice, it is not considered as likely to be a critical case as in other Arctic areas.

4.2.6 Auxiliary Systems

4.2.6.1 Cargo Transfer and Resupply

During the winter months, the structure may be surrounded by a rubble pile of significant areal extent. Resupply of the platform during the winter months will require careful evaluation and is a critical consideration in terms of confidence in any development plan. The platform can be resupplied during the summer months with conventional supply boats anchored close to the structure and off-loaded by structure cranes or stiff legs. During the winter months, however, supply boats may not be feasible and some kind of rough terrain vehicle may be needed to resupply the structure.

Boat landings for summer months would be desirable to facilitate crew and cargo transfers; however, they must be removable for winter operations to avoid significant damage from the rubble pile accumulations. Perhaps a suspended boat landing similar to those used in the Santa Barbara Channel is possible. Because the platform legs are armored at the water surface, barge bumpers will probably be unnecessary.

.2.6.2 Pipeline Risers

Moderate size pipeline risers may be installed through J-tubes incorporated into the large diameter legs, and exiting the structure beneath mudline. Similar types of J-tubes were incorporated in the Shell Middle Ground Shoal platforms in the Lower Cook Inlet. The J-tubes should be located in the rear legs to avoid any possible interference with the well conductors.

4.2.6.3 Disposal and Pump Casing

Disposal and pump casing also must be incorporated in some protected environment, such as internal to the main platform legs with disposal through a "J" tube near the mudline.

4.2.6.4 Corrosion Protection

The cathodic protection system must be designed to ensure long term servicability. Typical sacrificial anodes are not thought to be practical because of their exposed position during the winter months and the associated damage from the rubble pile. Some type of low profile impressed current system is possible, although currently available systems may be susceptible to significant ice damage.

Consideration may even be given to eliminating cathodic protection and instead relying on epoxy coating and a "corrosion allowance" of steel plate thickness. For example, the dense epoxy developed by Warsilla Shipyards in Sweden or the dense polyurethane used on the USGS icebreakers may be more suitable than cathodic protection.

4.2.6.5 Diver and Emergency Egress

In the Shell Middle Ground Shoal platforms, diver access tubes and hatches were provided for under-ice access for divers. This is thought to be a reasonable solution for this type of structure, due to the large diameter legs.

Emergency egress for the winter months must be carefully planned and perhaps some egress system internal to the main platform legs, is possible. Typical external stairways down to the water line may be impractical for this location.

4.3 St. George Basin

4.3.1 Base Case

4.3.1.1 Design Conditions

Base case conditions for the St. George Basin are 450 feet of water, 300,000 barrel per day production, medium clay soil, and the environmental conditions listed under "design basis" in Table 4-1. To produce 300,000 barrels of oil per day, a total payload of 45,900 tons is estimated. To accommodate the 48 wells assumed for 300,000 BPD production, the drill legs are 24 feet in diameter, providing a minimum well spacing of approximately 3-1/2 feet center to center if the wells are arranged in two concentric circles.

4.3.1.2 Results of Analyses

Analytical Model

A rough finite element model of the 4 plus 4 template was developed using program INTRA-WACS. Figure 4-4 is a sketch of the model. Main jacket legs and most major braces are explicitly represented by beam-column or truss elements having distributed wave loading capability, but simplified modeling is utilized for the decks, the horizontal inter-frame braces, the lower bay of the jacket and the foundation.

Beam-column elements at elevation +65 feet represent the weight and bending and axial stiffness of the deck support girders along rows 2, 3, A and B; and a stiff 2-D plane stress element represents the in-plane stiffness of the deck equipment module floors. The 91,800 kip payload is divided equally among the four legs. Bending moments are applied at the tops of the legs to account from some unbalanced bending moment from the deck support girders. Lateral wind forces are applied at these same nodes, along with vertical force couples equaling the overturning forces due to wind.

Note that the skirt piles and associated framing in the lowest jacket bay are not explicitly modeled. Thus the results only provide information on the pile group forces associated with each main leg of the jacket.

Dummy piles have been sized to give the mudline stiffnesses listed for medium stiffness soils (see Section 4.2.4), assuming a cluster of four 7.5 foot diameter piles at legs A1, A4, B1, and B4 and one pile at legs A2, A3, B2, and B3. It was subsequently recognized that one of the skirt piles clustered at each corner would also participate in resisting loads at legs A2, A3, B2 and B3. Thus, a more realistic distribution of stiffness might have been 3.7 parts at legs A1, A4, B1 and B4 to 1.3 parts at legs A2, A3, B2 and B3. The analyses were not rerun with revised foundation stiffnesses because such changes would have little impact on conclusions. However, the participation of the skirt piles in resisting interior leg loads has been recognized in choosing pile penetrations.

Results

Table 4-2 summarizes the global loads on the platform, listing mudline shears, overturning moments, torsions and required pile group capacities for various in-place load combinations. Generally, the most severe loads were produced by design storm conditions; lateral shears for storm ranged between approximately 11,000 and 13,000 kips, depending on the direction of wave approach. Note that the listed values in Table 4-2 are maxima for any load directions that were considered. For example, the listed X-shear for load combination 2 is associated with loading along the X-axis, the Y-shear is associated with loading along the Y-axis, and the required pile group capacities at corner legs A1 and A4 are demanded by storm loads acting along the diagonal of the platform.

Additional insight to the platform response is offered by sketches showing nominal element stresses for some of the more critical load combinations and load directions (see Figures 4-5 through 4-14). These sketches show axial stresses and maximum bending stresses at either member end, neglecting joint cans. The member end forces were determined in analyses using an earlier iteration on member sizes, but these forces were converted to stresses using the final section properties shown in Drawing 102. Therefore, the calculated internal force distribution is only accurate to the order of approximately ten percent. This is felt to be adequate accuracy for the purposes of this study.

The computed stresses are generally within permissible limits, indicating the feasibility of the proposed preliminary design. Remembering that basic allowable stresses are permitted for operating conditions, a one-third increase in stresses is permitted for design storm or ice, and a seventy percent increase is permitted for strength seismic, one can note the controlling load combinations for different portions of the platform. The deck girders, interior legs, exterior legs above elevation -230', and the vertical bracing in the upper two bays on rows 2 and 3 are controlled by load combination #1 (dead plus live plus buoyancy). Additionally load combination #1 controls the horizontal braces at elevation -125' on rows A and B and the braces running from the interior legs at -30' to the exterior legs at -125' on rows A and B, all of which act to transfer some of the deck load to the exterior legs. The lower portions of the exterior legs and most of the vertical bracing are controlled by storm loading (combination #2). Horizontals are not typically loaded heavily by in-place conditions; they are controlled mostly by geometric limits or transportation and launching stresses. They also provide alternate load paths through the structure, helping provide the redundancy necessary for successful performance during overloads, such as extreme earthquake.

The structure's thirteen lowest natural vibration frequencies and mode shapes were computed. Table 4-3 lists the periods and key characteristics for each vibration mode, and Figures 4-15 through 4-19 show the deformed shapes for several modes. At 4.2 and 3.7 seconds, the structure's two fundamental lateral sway modes are in a frequency range where significant wave energy is anticipated. Thus, fatigue will probably have major influence on the design. Conversely, the fundamental periods are beyond the range where significant seismic energy is concentrated; and the results of a response spectrum analysis (Figure 4-20) indicate that seismic stresses are not large enough to significantly influence the design.

Note that the frequency analysis was based on a medium sand profile rather than the clay profile which was chosen as the base case for St. George Basin. This was done for conservatism. The sand profile is somewhat stiffer, and therefore should result in slightly lower periods and higher spectral ordinates than would the clay profile. It is estimated that the clay soil profile would increase the fundamental sway periods by no more than ten percent, and higher periods by even less.

Figures 4-21 and 4-22 indicate the stresses caused by load combination #5 (ice slam). Ice slam appears to be important for local design of the leg shell, but, comparing against Figures 4-4 and 4-5, it is a relatively minor perturbation on normal operating stresses.

4.3.1.3 Preliminary Design

Jacket

Member sizes and key features for the base case design for St. George Basin are similar to those presented in Drawings 101 through 102. The template is designed to be launched at sea from a barge; but the large diameter drill legs seem to make it convertible to a self-floater. The large diameter legs (drill legs and legs B2 and B3 above elevation -126') are internally stiffened both axially and circumferentially, have internal diaphragms with conductor penetrations at the horizontal framing levels, and additionally have double wall sections with grout in the annuli from elevations +20' to -30', to resist ice impact. Internal stiffeners are designed to preclude local buckling of the leg shells and resist full hydrostatic pressure during upending. The backside legs, B2 and B3, are large diameter at the two upper bays so that the stiffness of the moment frames is balanced to preclude excessive torsion about a vertical axis. Major appurtenances on the template are mudmats with supporting knee braces back to the legs, skirt pile guides with knee braces back to the legs to resist the dynamic weight of a skirt pile if it is not stabbed accurately during installation, corrosion protection, and a deep launch cradle to make up the difference between the changing radii at the launch legs. Alternatively, eccentric leg axes could be employed at the launch legs. Fifteen-foot leg extensions are provided at the base of the jacket to give additional support during touchdown.

Decks

Massive deck girders are required to support the heavy deck loads and complete the moment frames at the top of the platform. Drawing 104 shows a suggested layout and possible girder details. The equipment modules can be supported primarily by the box girders on rows 2 and 3. The inside edges of the modules are supported by a middle girder which is in turn supported by transverse box girders. For the suggested girder sections, it was calculated that the maximum deflections on rows 2 and 3 will be less than 1.25 inches under full deck load, and the maximum deflection at the middle girder will be approximately 5 inches under full deck load, including the deflection of its supporting transverse girders. These deflections should not pose serious problems, if properly handled in the module designs.

The proposed deck support structure is too heavy to be lifted in a single piece by any available derrick barge. Two options are available:
(1) fabricate the deck support frame in two pieces which can be set separately at the site and then connected structurally, or (2) integrate the deck support frame with the jacket, such that they are fabricated, transported and installed as a unit.

The first option is complicated by the fact that the deck is supported by only four legs. Thus, temporary shoring or moment-resisting stabbing guides are needed for stability if the deck support structure is to be set segmentally. Elevation E1 on Drawing 104 suggests a possible shoring scheme. One half of the deck support structure could be outfitted with temporary braces and two temporary legs which would mate with temporary jacket columns. The other half of the deck support structure could be slung at an angle so that first its two legs would stab the jacket and then the ends of its girders would roll down onto seats in the girders above the temporary columns. The box girders would then be spliced together, and the temporary braces cut loose and removed. This scheme requires extensive field welding and careful attention to fit-up problems.

The second option, an integrated deck substructure and jacket structure, also poses some problems. The cantilever portions of the girders on one side of the platform could not be installed until after loadout, and during tow they would drag in the water. Additionally, the massive deck support girders would be the first elements to enter the water during launch, tending to slow the jacket and to keep the dive angle shallow, which could hinder clean separation. An integrated deck support structure would eliminate the need for a temporary pile guide deck during pile installation, but would necessitate the use of shorter pile sections because the pile add-ons would be made at a higher elevation. The cost estimate and schedule are based on the integrated approach.

Foundation

The foundation proposed for the base case in the St. George Basin comprises six main piles and twelve skirt piles, all 90 inches in diameter. Additionally, the well drive pipes are assumed to function as piles at legs A2 and A3.

The skirt piles are clustered three around each corner leg and slightly interior to the column rows. Each skirt pile is spaced at least 20 feet (2.67 pile diameters) from neighboring piles so that pile capacities will not be reduced excessively by group effects. Since the inner legs (rows 2 and 3) are spaced widely to promote ice clearing and tow stability, the skirt piles on rows A and B fall almost midway between a corner leg and an inner leg. Thus, these piles carry loads from both the inner and outer legs. The skirt piles along rows 1 and 4 only carry loads from rows 1 and 4.

The following pile penetrations are estimated for medium clay soil.

Piles A1 and A4	390 feet (19400k)
Piles B1 and B4	300 feet (10000k)
Piles B2 and B3	340 feet (14300k)
Skirt Piles	330 feet (13200k)

These penetrations are based on the design basis pile capacity curves from Figure 4-3 and assume 85 percent efficiency for the pile clusters at each corner of the platform. The resistance of the three skirt piles at each corner cluster is assumed to be devoted 2.7 parts to loading from the corner legs and 0.3 parts to loading from the inner legs. The total installed pile height, including above-mudline height in the jacket legs and skirt pile sleeves, is estimated to be 10,300 feet. A minimum wall thickness of 1.5 inches is assumed for drivability.

4.3.1.4 Material Estimation

Tables 4-4 and 4-5 show the material estimates for the jacket, deck support structure and piling. The 17400 ton jacket consists of 6000 tons of steel in the legs, 5600 tons of steel in other primary structural elements and braces, 1000 tons for mudmats, 2500 tons for other appurtenances, and a 20 percent allowance (2300 tons) for secondary braces, thickening at joint cans, miscellaneous stiffeners and mill tolerance. Total piling weight, 9030 tons, is estimated as the product of nominal pile weight per foot times installed pile height plus 20 percent extra to account for thick wall sections near the mudline, pile stabbing guides and cut-off allowances, plus 260 tons for a follower for driving the skirt piles. Table 4-5 also lists the estimated weight for the deck support structure. The listed weights do not include structural steel in the deck equipment modules. The weight of module structural steel is included in the 45900 ton payload.

4.3.1.5 Construction Planning

Fabrication

There are several fabricators on the West Coast of the United States or in Japan who can accomplish the proposed design (see Table 4-6). The large diameter drill legs present some fabrication problems. They are too large to be fabricated from a single circumferential plate and will require a minimum of two longitudinal weld seams. Additionally, the roll-ups involving the drill legs will be very heavy--approximately 6.5 tons per foot or 3200 tons total for rows 2 or 3. Thus, single piece roll-up will probably not be possible for rows 2 and 3. These difficulties could result in somewhat higher unit prices than normal, and are reflected in the cost estimates.

Transportation

There are five launch barges worldwide which can accommodate the proposed jacket. It is estimated that such barges require a draft of approximately 25 to 30 feet for end loadout. If the deck support structure is built integrally with the jacket, the deck box girders must overhang aft of the barge, and additional time will be required after loadout to install cantilever portions of the box girders which will drag in the water. This extra effort during loadout is compensated by benefits during installation at the site: an integrated deck structure eliminates the need for a temporary pile guide deck and it may reduce the derrick barge time at the site because deck structure need not be set.

The tow distance is approximately 2500 statute miles from either San Francisco Bay or from Japan. Using two 9000 horsepower tugs to provide a tow rate of approximately 2.5 knots, the tow could be completed in approximately 36 days.

Material barges will be required for the piling, conductors and deck substructures. One 100' x 400' barge should suffice for the piling and conductors. An additional barge will be required if the deck structure is not fabricated integrally with the jacket. Each barge will demand one 9000 horsepower tug and can travel at a rate of approximately 8 knots.

Installation

The jacket will be launched top first without need for any auxiliary buoyancy. The feasibility of launching with the deck support structure in place has not been confirmed.

Major equipment requirements for installation include:

- One semi-submersible derrick barge with a 2000 ton revolving crane.

- A pile hammer (and back-up) with a rated energy in excess of 1,000,000 foot-pounds.

- Grouting equipment and supplies for the jacket legs and the skirt pile sleeves.

- Monitoring and control equipment for remote control ballasting during upending, stabbing skirt piles, et cetera.

- Diving spread.

- RCV spread.

4.3.2 Sensitivities to Variations in Environmental and Operational Parameters

4.3.2.1 Water Depth Variation

The sensitivity of the base case St. George Basin design to changing water depth has been evaluated by extrapolating the analytical results for the base case. Table 4-7 lists the estimated forces for two of the most influential load case combinations (combination 1 = dead plus live plus buoyancy and combination 2 = storm) at the design water depth of 450 feet and at upper and lower water depths of 600 and 300 feet. It is anticipated that lateral base shears and total vertical reactions will change very little for changing water depths. Lateral shears are dominated by wave forces, and the preponderance of wave force is in the near surface water. The other components to lateral storm shear are wind and current. Wind force does not change with water depth, but current force does change because the projected area of the structure that is exposed to current changes. Total vertical force will change slightly due to the changing submerged weight of a taller or shorter jacket. This change is small proportionally because the total vertical force is so highly dominated by deck payload. Of course, base overturning moment and corner pile group forces are highly dependent on water depth. Nearly a 50 percent increase or decrease in overturning is extrapolated for the upper and lower water depths. Approximately a 20 percent change in corner pile forces is calculated.

The proposed base case design is suitable for changing water depths from 300 to 600 feet. The design can be modified by adding or subtracting one vertical bay, stretching or compressing the distance between horizontal framing levels, and adjusting pile penetrations. It is assumed that the dimensions at the top of the jacket and the leg batters remain constant; therefore the base of the jacket is slightly bigger or smaller in plan for different water depths. In almost all other aspects, including member sizes, the structure remains highly similar to the proposed base case design. Fatigue is expected to become more of a problem in deeper water (due to a more flexible structure); therefore, the jacket contingency is increased from 20 to 30 percent.

Table 4-8 summarizes the estimated structural weight for three water depths that have been studied. Total structure weight ranges from 24,370 to 36,470 tons, 17 percent less and 24 percent greater, respectively, than the base case design. The variation in jacket weight is minus 21 percent or plus 33 percent.

The construction equipment necessary for the upper and lower water depths could be virtually identical to that described for the base case.

4.3.2.2 Production Rate Variation

A reduced production rate of 100,000 barrels of oil per day has been studied to evaluate the influence of reductions in deck payload and number of wells. It is estimated that 100,000 BPD production can be achieved with a total deck payload of 31,550 and 30 well conductors, compared to 45,900 tons and 48 conductors for the 300,000 BPD base case.

Table 4-9 presents the mudline reactions and required pile capacities for a 4 plus 4 template producing 100,000 BPD in 450 feet of water in

St. George Basin. The results for load combination #1 are from computer analysis; results for other load combinations are extrapolated.

The structure for 100,000 BPD production experiences substantially less vertical load and less pile load than the base case--approximately 25 percent less; this is primarily due to the reduced deck payload. The structure also experiences somewhat reduced lateral shears. Since only 30 wells are planned, the diameter of the drill legs can be reduced from 24 to 15 feet and still maintain a minimum spacing of roughly 3.5 feet between wells. This reduces the projected area loaded by waves, current and ice.

The major changes in the base case structure permitted by a reduction to 100,000 BPD production are:

- Reduction of the diameter of the drill legs from 24 to 15 feet.
- Minor reductions in the size of deck girders, jacket vertical bracing and mudmats.
- Reduction in the required penetrations of the piles into the soil. A preliminary design based on the medium clay soil profile has penetrations of 300 feet for the skirt piles, 350 feet for main piles A1 and A4, 275 feet for main piles B1 and B4, and 300 feet for main piles B2 and B3.

The estimated structure weight is 15,500 tons for the jacket (Table 4-10), 8560 tons for the piling, and 2430 tons for the deck support structure (Table 4-11). This is a 10 percent reduction from the total weight of the base case structure. Construction planning for the lower production rate structure would be similar to that for the base case.

A primary sensitivity to number of wells is the limit on total wells to about 48 as discussed in Section 3. This implies more than one structure for a larger number of wells.

4.3.2.3 Variation in Environmental Load

Table 4-1 lists assumed upper and lower bound environmental loading conditions. The assumed range in seismic loading is well below the point of becoming a significant influence on the structural design. Likewise, the range in ice slam pressures does not indicate strong influence on the global design, although ice slam is of course important for the local design of the armored leg sections which penetrate the water surface. Loading from a rafted ice flow could match the influence of storm loads on the structure if it approaches its upper bound value (approximately 11000 kips lateral shear for load combination 5) while the design storm decreases to its lower bound value. Still, the environmental loading likely to have strongest influence on the structural design appears to be waves, both design storm and long-term fatigue. A supplemental computer run was made to calculate the response of the base case design to the upper bound wind, wave and current acting in the plus X direction. The gross maximum mudline reactions from load combination 2 with upper bound wind, wave and current were:

X shear	13,500 kips
Y shear	352 kips
Vertical	108,100 kips

X moment	198,400 kip-feet
Y moment	4,490,000 kip-feet
Torsion	127,000 kip-feet

The calculated range in loads from lower to upper bound storm conditions suggests very minor variations in structure weight, really too small to be perceivable at the level of this study.

A more important variation is the possibility of a rafted ice floe bridging between the legs and loading the entire cross section piercing the water surface. If the proposed structure is to remain feasible, it should be confirmed in final design that this will not occur. Should it be concluded that bridging ice can occur, the structure will need to be stouter. The base case design for the Navarin Basin is controlled by bridging ice and indicates the kinds of strengthening that could be required.

4.3.2.4 Variation in Soils

Two types of variations in soils have been considered. First, both clayey and sandy soils are considered. Second, for each soil type a range of soil shear strength profiles is considered. These ranges are indicated by Figures 4-1 and 4-2.

The operating and storm loads which control the base case design for St. George Basin are essentially independent of foundation properties. Thus it is appropriate to use the required pile group capacities from Table 4-2 to design pile penetrations for different soil types and stiffnesses. Estimates of total installed pile length for various soil profiles are as follows:

	Lower Bound	Design Basis	Upper Bound
Clay	11800	10300	9000 feet
Sand	6300	6100	5900 feet

The only change in the material estimation for the base case due to changing soil conditions is in the piling. Table 4-12 presents weight estimates for the structure in various soils.

4.4 Navarin Basin

4.4.1 Base Case

4.4.1.1 Design Conditions

Base case conditions for the Navarin Basin are 450 feet of water, 300,000 barrel per day production, medium clay soil, and the environmental conditions listed under "design basis" in Table 4-13. Again, 45,900 tons of payload are assumed and two 24-foot diameter drill legs are provided to contain 48 wells.

4.4.1.2 Results of Analyses

The same model described for the St. George Basin (Figure 4-4) has been used to analyze the structural response to conditions at the Navarin Basin. Ice loads become dominant at the Navarin Basin because ice floes are assumed to span the entire projection of the structure. Figure 4-14 summarizes the gross forces and required pile group capacities for five key load case combinations that have been considered. Clearly, the rafted ice floe loading in combination #4 is dominant, particularly for the foundation. The gross lateral shears for this combination are approximately 14 percent greater than the worst loading (storm) at the St. George Basin, and overturning moments are roughly 50 percent greater.

Figures 4-23 through 4-28 show nominal member stresses for load combination #4 with the design ice floe and wind acting diagonally across the platform. Stresses under vertical operating loads are similar to those shown in Figures 4-5 through 4-8. Comparing these two sets of figures, one can note that load combination #4 controls the size of virtually all members except the inner legs between elevations (-)30' to (-)125' and some of the horizontal braces.

Structure dynamic characteristics are expected to be very similar to those described for the St. George base case structure. Relatively long natural vibration periods will demand careful design of the joints to avoid fatigue damage. Seismicity in the Navarin Basin is even less than that in the St. George Basin and has no major influence on the design.

4.4.1.3 Preliminary Design

Jacket and Deck

The jacket and deck structure for the base case in the Navarin Basin are very similar to those for the St. George Basin, but with somewhat heavier sections in some of the vertical bracing and legs. Drawings 101 through 104 indicate a preliminary design for the Navarin Basin.

Foundation

Due to the greater overturning moments, the base of the jacket and pile foundation must be stronger than those proposed for the St. George Basin. The same pile layout and pile diameters as used at St. George are proposed for Navarin, i.e., six main piles and twelve skirt piles clustered three around each corner leg. Then for the same medium clay soil profile, greater pile penetrations are estimated:

Piles A1 and A4	430' (25,500 k)
Piles B1 and A4	355' (15,900 k)
Piles B2 and B3	360' (16,400 k)
Skirt Piles	380' (18,800 k)

Again, these penetrations are based on the pile capacity curves from Figure 4-3 and assume 85 percent efficiency for the corner pile groups. The total installed pile length is estimated to be 11,150 feet, and a minimum wall thickness of 1.75 inches is assumed.

4.4.1.4 Material Estimation

Tables 4-15 and 4-16 show material estimates for the jacket, deck support structure and piling for the base case design in the Navarin Basin. At 18,000 tons, the jacket is only slightly heavier than that proposed for St. George; the increase is concentrated in the vertical bracing and shells of the legs. The deck support structure for the Navarin Basin is the same as that for St. George, the piling is 25 percent heavier due to increased penetrations and greater wall thickness. Total pile weight is 11,270 tons.

4.4.1.5 Construction Planning

The base case structure for the Navarin Basin can be installed using equipment and procedures similar to those discussed for St. George Basin (see Section 4.3.1.5). The tow distances are approximately 2500 statute miles from Japan or 2900 miles from San Francisco Bay. Using two 9000 horsepower tugs at a speed of approximately 2.5 knots results in tow durations of 36 and 42 days from Japan or San Francisco Bay, respectively. The deep pile penetrations will probably demand a hammer with a rated energy of roughly 1,500,000 foot-pounds.

4.4.2 Sensitivities to Variations in Environmental and Operational Parameters

4.4.2.1 Water Depth Variation

Load combination #4 (dead plus live plus buoyancy plus rafted ice plus wind) is expected to control the platform design for water depths in the range of 300 to 600 feet. These loads are nearly independent of water depth, except for small changes in vertical loading due to a shorter or taller jacket. Of course, pile loads decrease or increase with base overturning moment, which varies linearly with water depth.

Table 4-17 shows the estimated platform weights for three different water depths. The total estimated weights are 26,790, 32,250, and 42,860 tons for water depths of 300, 450, and 600 feet respectively. The deck support structure can be the same for all three structures. The jacket for 300 feet of water is similar to the base case but with one fewer horizontal framing level, and with somewhat shallower pile penetrations. A different pile layout is proposed for the 600' water depth structure in order to avoid excessively long pile penetrations. Figure 4-29 suggests a possible foundation layout utilizing six main piles and eighteen skirt piles. In addition to adding steel weight for a taller structure and greater overturning moments, this foundation scheme adds steel weight due to more pile sleeves and interconnecting framing.

4.4.2.2 Production Rate Variation

Table 4-18 presents the gross platform forces and required pile group capacities for various load case combinations on a 4 plus 4 template for 100,000 barrel per day production in the Navarin Basin. The deck payload is assumed to be 31,550 tons, and 30 wells are assumed to be contained in two 15-foot diameter legs. Load combination #4 remains dominant, with the same lateral shears and overturning moments as the base case but with somewhat reduced vertical loads due to the reduced payload. The base case lateral shears due to ice are unaffected because the entire projection of the structure at the water surface is loaded, not just the legs.

Tables 4-19 and 4-20 summarize the estimated structure weight for 100,000 BPD production. At 16,100 tons, the jacket is approximately 11 percent lighter than the 300,000 BPD base case. The total weight of jacket, piling and deck support structure is 29,590 tons; this is 8 percent lighter than the base case.

4.4.2.3 Variation in Environmental Loads

Table 4-13 indicates a range for environmental conditions in the Navarin Basin. Since the load case combination involving rafted ice loading controls the base case design, this sensitivity study has focused on variations in rafted ice.

Assuming that the structural responses to ice and wind vary linearly with ice load, upper and lower bound response can be estimated from the base case results by proportioning the difference in response for load combinations #4 and #1. Table 4-21 shows the estimated values.

Rough estimates of platform weight for varying ice loading conditions in the Navarin Basin are presented in Table 4-22. For the upper bound loads, the required pile group capacities are believed too large to be accommodated in medium dense clays by the foundation layout proposed for the base case. A layout utilizing 6 main piles and 18 skirt piles, similar to that shown in Figure 4-29, is believed to be more feasible. The estimated structure weight for upper bound ice is based on such a foundation layout. The variation in total structure weight is minus 3 percent for lower bound ice loading and plus 18 percent for upper bound, relative to the base case.

4.4.2.4 Variation in Soils

The magnitudes of the loads which control the platform design in the Navarin Basin do not vary significantly with soil type. Therefore, the base case designs for the deck support structure and jacket are applicable in different soils. Of course the range of application is limited to the feasible range for the chosen foundation layout.

For the loads given in Table 4-14 and the base case foundation layout of 6 main piles and 12 skirt piles, the estimated installed pile lengths required for the various soil profiles are:

	Lower	Design	Upper
Clay	13300	11150	10000
Sand	6700	6500	6300

The above estimates demand some large individual pile capacities and deep penetrations. It will be important to confirm their feasibility with drivability analyses.

Table 4-23 shows estimated structure weights for various soils in the Navarin Basin.

4.5 North Aleutian Shelf

4.5.1 Base Case

4.5.1.1 Design Conditions

For the North Aleutian area, the base case conditions are 300 feet of water and a production rate of 300,000 barrels of oil per day. The latter requirement is presumed to demand a deck payload of 45,900 tons and 48 well conductors (see Sections 2.3.2 and 2.7). These wells can be accommodated inside two 24-foot diameter legs if they are arranged in two concentric circles with a circumferential spacing of approximately 3.5 feet center-to-center. The environmental conditions for the North Aleutian Shelf are listed in Table 4-24.

4.5.1.2 Results of Analyses

A limited number of structural analyses have been performed using a finite element computer model. This model is similar to the one described for the St. George Basin (in Section 4.3.1.2), but has one fewer horizontal framing level and is 150 feet shorter. The lateral dimensions at the top of the jacket and the leg batters are the same, so for a shorter structure the base dimensions do not spread to as great an extent. The foundation modeling is the same as for St. George Basin.

Calculated or extrapolated loads for five key in-place load case combinations are given in Table 4-25. Results for the first three load combinations are directly from computer analyses; results for load combinations 4 and 5 are extrapolated from previous analysis for the deeper water at

St. George Basin. Lateral shears due to wind are extrapolated by assuming they vary as the square of the ratio of design wind velocities, and shears due to ice vary as the ratio of design ice pressures.

Storm conditions induced the greatest lateral shears, followed by rafted ice plus wind and then strength requirement earthquake. Figure 4-30 shows the assumed design spectrum for earthquake. With a peak ground acceleration of 0.24g, the North Aleutian area has the highest seismicity of the three study areas. Even so, calculated earthquake forces do not appear significant for the proposed design. Figures 4-31 through 4-37 show nominal member stresses for load combinations 1, 2 and 3. Stresses induced by earthquake, gravity and buoyancy, which are permitted to reach 1.7 times the basic allowable stresses in API RP 2A, are observed to be less critical than storm, rafted ice or normal gravity stresses. The vertical bracing and corner piles are controlled mainly by load combinations 2 and 4, storm and rafted ice. Jacket legs and interior piles are controlled primarily by maximum gravity and buoyant loads. One can note that the stresses are within the range of feasibility.

Earthquake loads were computed in a response spectrum analysis. The first step in such an analysis is calculation of the structure's natural vibration frequencies and mode shapes. Table 4-26 lists the structure's thirteen lowest frequencies and briefly describes the primary characteristics of response. The calculated periods are not as long as those for the deeper base case water depth at St. George and Navarin Basins, and thus the North Aleutian structure will probably be less susceptible to wave-induced fatigue damage. However, API RP 2A does recommend a fatigue analysis for structures with a fundamental period exceeding 3 seconds.

With a fundamental period of 3.2 seconds and the harsh wave climate anticipated for this area, it is likely that some joints in this structure will need strengthening to avoid low fatigue lives. Direct analysis of this is beyond the scope of this project, but the material estimate allows some contingency for fatigue strengthening.

4.5.1.3 Preliminary Design

The proposed preliminary design for the North Aleutian jacket is indicated by Drawing 105. Horizontal framing is similar to that shown in Drawing 103. The deck support structure is the same as that for the St. George and Navarin Basins, and is shown in Drawing 104.

Due to the shallower water depth and the presence of sandy rather than clayey soils, the pile penetrations required for the base case design are substantially less than those needed at St. George or Navarin Basins. Using a pile layout of six main piles and twelve skirt piles clustered three around each corner leg, the loads in Figure 4-49 require the following pile penetrations in "medium" density sand:

Piles A1 and A4	105 feet (14,200 k)
Piles B1 and B4	70 feet (7,100 k)
Piles B2 and B3	110 feet (15,000 k)
Skirt Piles	85 feet (10,000 k)

4.5.1.4 Material Estimation

Tables 4-27 and 4-28 list steel tonnage for the various components of the proposed platform. The jacket weight is 13,500 tons. Total structure weight, exclusive of deck module steel, is 20,830 tons.

.5.1.5 Construction Planning

Likely fabrication sites for the structure are Japan or the western coast of the continental United States, particularly the San Francisco Bay Area. Necessary installation equipment will be similar to that described in Section 4.3.1.5.

4.5.2 Sensitivities to Variations in Environmental and Operational Parameters

4.5.2.1 Water Depth Variation

Table 4-29 shows the maximum loads and required pile capacities extrapolated from the base case results. The listed values are maxima among load case combinations 1, 2 or 4, the three load combinations which exert significant influence on the base case design. Base overturning moment and corner pile loads have substantial sensitivity to changes in water depth; base lateral shear and gross vertical load have rather mild sensitivity to changes in water depth.

Table 4-30 lists the estimated weight of steel template structures and piling for three water depths. The estimate for the lower water depth, 200 feet, is based to a large degree on judgment. For this case, the 4 plus 4 template is probably not the most promising type of structure; a tower type structure with supplemental skirt piles may be a better candidate for water this shallow. The estimate for the structure in 200 feet of water assumes a four-legged tower with three supplemental skirt piles at each corner. The range in structure weights is 15,510 tons to 22,800 tons, 27 percent less and 8 percent greater respectively than the base case design.

4.5.2.2 Production Rate Variation

Table 4-31 shows extrapolated loads and required pile group capacities for a 4 plus 4 template suitable for a reduced production rate of 100,000 barrels per day. It is assumed that this production rate demands a deck load of 31550 tons and two 15-foot diameter legs to contain 30 wells. The listed loads are extrapolated maxima among load combinations 1, 2 or 4, the three most influential load combinations. Lateral ice loads are extrapolated from the base case results by ratioing by 15/24, the ratio of 10,000 BPD leg diameter to base case leg diameter. Storm loads are factored from the base case results by multiplying by $(0.5 + 0.5\frac{15}{24})$, assuming roughly half the load comes from the large diameter legs.

The estimated weight of the jacket for 100000 BPD production on the North Aleutian Shelf is 12,600 tons (see Table 4-32), and the deck support structure is estimated to weigh 2430 tons (see Figure 4-11). Pile penetrations can be approximately 17 percent less than those for the base case, yielding an estimated pile weight of 4120 tons. The total structure weight is 19,150 tons, 8 percent less than the base case.

4.5.2.3 Variation in Environmental Loads

Both storm and rafted ice loading exert influence on the base case design. Thus, independent variation in one of these parameters is not perceived fully in the structural design because the other unchanging parameter becomes relatively more dominant and masks the influence of variations in the first parameter. Table 4-24 shows the assumed range of environmental loadings for the North Aleutian Shelf. The ranges in wave height, wind speed, and current speed indicate that lateral shears for load combination 2 might vary on the order of ten percent. This variation implies modest changes in the jacket vertical bracing, perhaps ±5 percent in weight. The greatest overturning moments are produced by rafted ice loading (load combination 4), so variations in storm loads do not require changes in the piling and lower portions of the legs. The lower bound for rafted ice loading, 55 ksi per foot of leg diameter, is 35 percent less than the design basis. However, its influence on the structure is limited because overturning due to design basis storm is only ten to twenty percent less than that for design basis ice. On the other hand, upper bound rafted ice loading has unmitigated influence. The upper bound rafted ice loading is nearly 30 percent greater than the design basis and induces nearly 30 percent greater overturning. This demands roughly a five percent increase in the penetration of the corner pile groups, or an increase in total pile weight from 4350 to 4410 tons.

4.5.2.4 Variation in Soils

Only sandy soils are expected on the North Aleutian Shelf. Figure 4-3 shows possible pile capacity curves for lower, design and upper bound sand densities. The base case pile penetrations increase approximately 10 percent and decrease approximately 7 percent for lower and upper bound soil densities, respectively. The estimated pile weights are 4490, 4350 and 4250 tons for lower, design basis, and upper bound soils, respectively.

4.6 Summary and Commentary

Preliminary designs of a 4 plus 4 steel template type offshore oil drilling and production platform have been prepared for the St. George Basin, Navarin Basin, and North Aleutian Shelf in the Southern Bering Sea. The eight leg template is supported by six main piles and twelve skirts piles. For base case conditions, 48 wells are contained inside two 24-foot diameter legs, where they can be better protected from ice and waves. The vertical bracing and four corner legs rise only to elevation -30 feet, so that the structure's profile is "clean" through the ice zone to minimize ice jamming. Box girders are used to support the deck equipment modules and complete a moment frame at the top of the structure. Drawings 101 through 105 indicate the proposed base case designs.

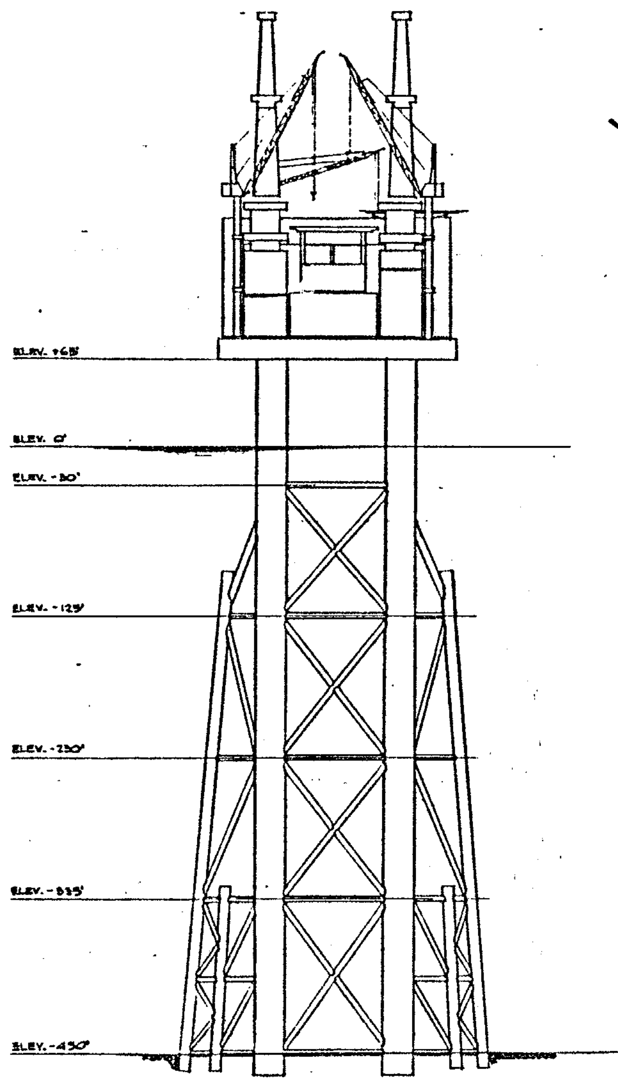
It is concluded that the 4 plus 4 template is structurally feasible for the St. George Basin, Navarin Basin, and North Aleutian Shelf. Due to heavy payloads and the severe environment, the designs for each basin are relatively heavy. The following table summarizes the estimated weights of jacket, piling and deck support structure for a base case design for each basin.

	St. George	Navarin	N. Aleutian
Water Depth	450 feet	450 feet	300 feet
Jacket	17400 tons	18000 tons	13500 tons
Piling	9030 tons	11270 tons	4350 tons
Deck Support Structure	2980 tons	2980 tons	2980 tons
Total Weight	29410 tons	32250 tons	20830 tons

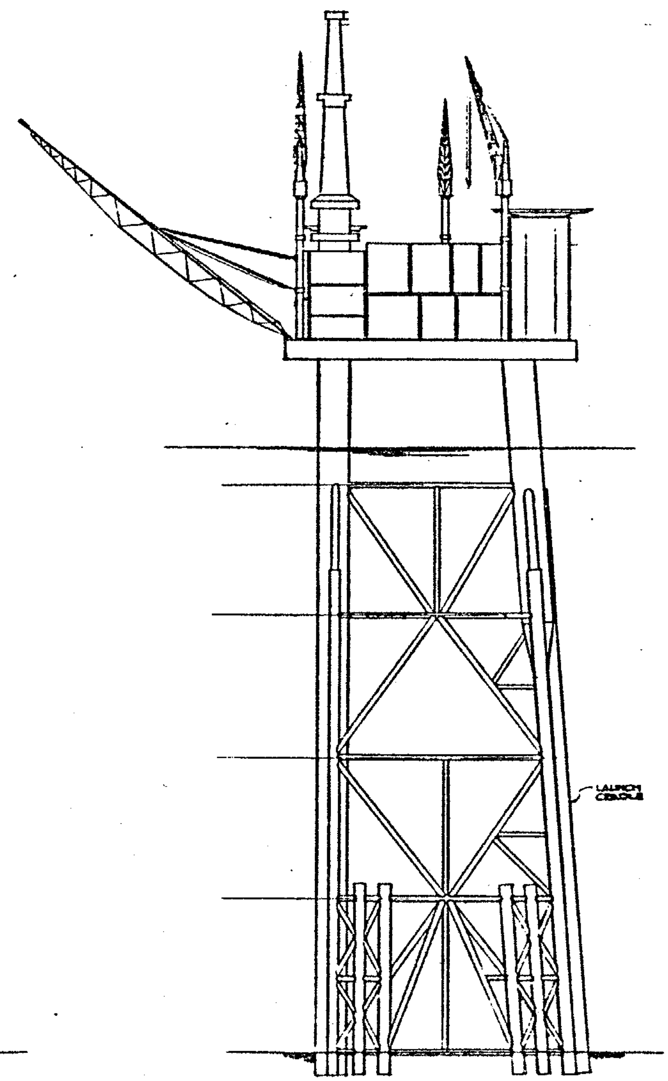
Figure 4-38 compares the estimated jacket weight for each base case design with some North Sea platforms. High strength, ductile steels (50 ksi yield strength and Charpy toughness of 25 foot-pounds) are proposed in order to help endure the cold and maintain good weldability by generally limiting wall thickness to about two inches. A twenty to thirty percent allowance has been made for thick walls at joint cans, mill tolerance, weld metal, secondary braces and miscellaneous stiffeners; this relatively large contingency is appropriate to accommodate designing the joints to resist fatigue and high punching shears. Fatigue is a concern because the structures are relatively flexible and a severe wave climate is anticipated; large punching shear stresses are likely because the leg diameters are large. Fatigue and punching shear analyses are beyond the scope of this study.

Fabrication will be complicated somewhat because the proposed special steels demand careful welding procedures. Additionally, the large-diameter legs, which are relatively heavy and have extensive circumferential and longitudinal stiffeners, will be expensive to fabricate. In other aspects, the proposed structures can be fabricated, transported and installed using reasonably conventional techniques.

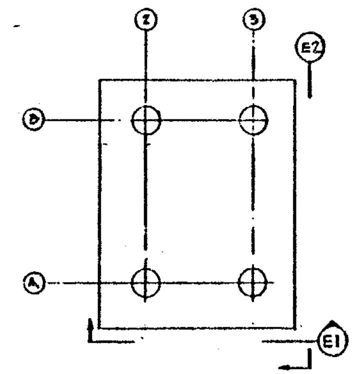
The proposed configuration is generally suitable for plausible ranges in water depth, production rate, environmental loads and foundation soils. Figures 4-39 through 4-41 and Table 4-33 show estimated structure weight as a function of these four variables. These figures are provided to indicate the sensitivity of the structure to changing conditions, and are intended to aid in developing bidding strategies for future offshore lease sales. The scope of this study is not sufficiently detailed to permit exact estimates for each condition. Furthermore, the 4 plus 4 template is undoubtedly not the optimum structural solution for all sites in the Southern Bering Sea. Thus, the estimates are indicative of approximate weight and trends in weight changes for varying conditions.



ELEVATION E1



ELEVATION E2



KEY PLAN

REVISION			Drawn	Scale
No.	Date	Description	DWP	
1			Checked SMW	Date
2				
3			Approved	Job No.
4				
5			Approved	Client Approv.
6				

PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

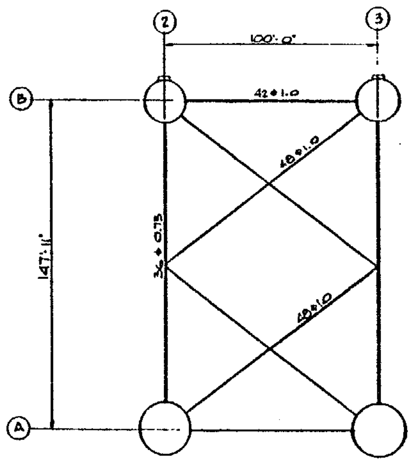
For

SOUTHERN BERING SEA
PRODUCTION SYSTEMS STUDY

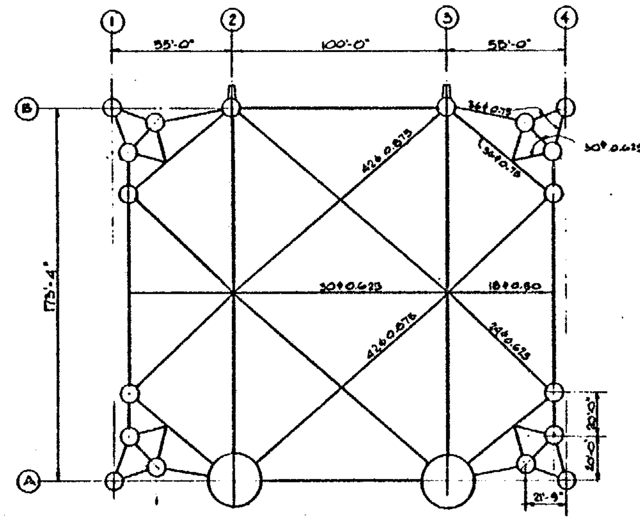
Drawing Title: TYPICAL ASSEMBLY ELEVATIONS OF STEEL TEMPLATE

Drawing No. 101

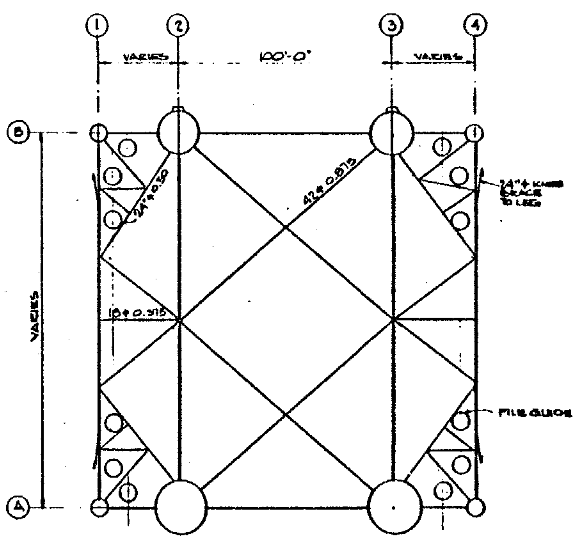
Rev.



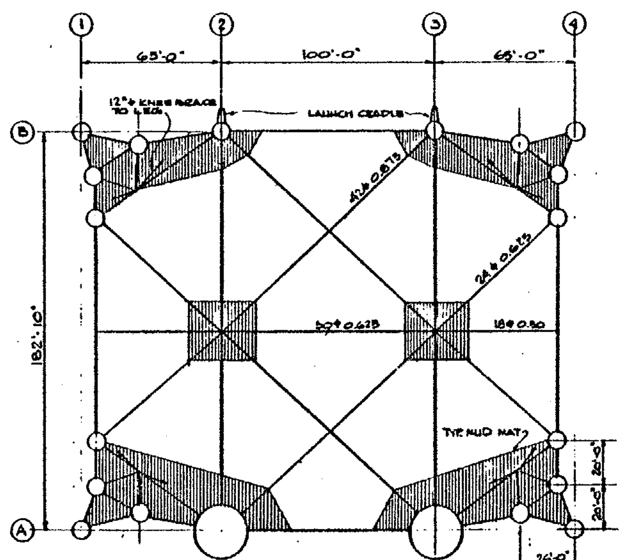
PLAN @ ELEV.-30'



PLAN @ ELEV. - 335'



PLAN @ ELEVS. - 125' & 230'



PLAN @ ELEV. -450'

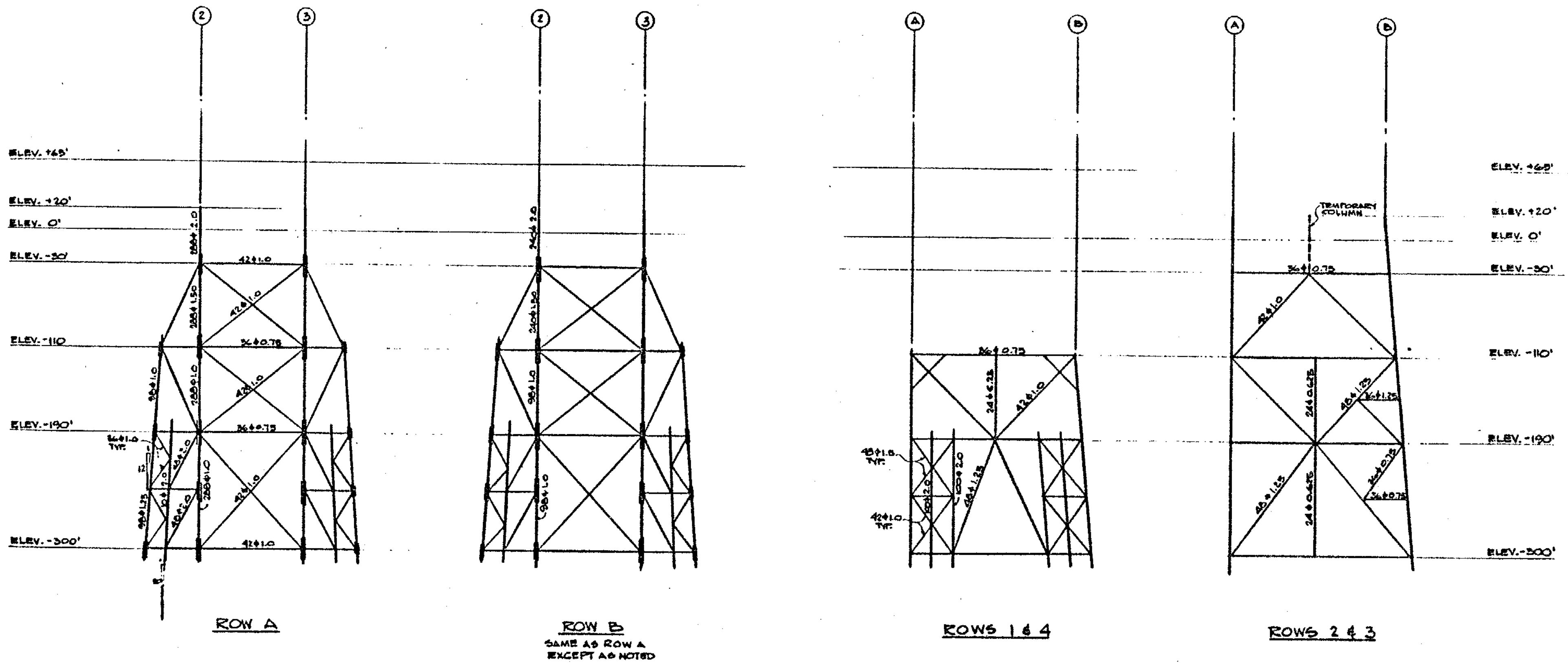
REVISION				
No.	Date	Description	Drawn OWP	Scale
1			Checked BMW	Date
2				
3			Approved	Job No.
4				
5			Approved	Client Approv.
6				

PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

For

SOUTHERN BERING SEA
PRODUCTION SYSTEMS STUDY

Drawing Title	Drawing No.	Rev.
STEEL TEMPLATE PLANS	103	



PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco — Houston

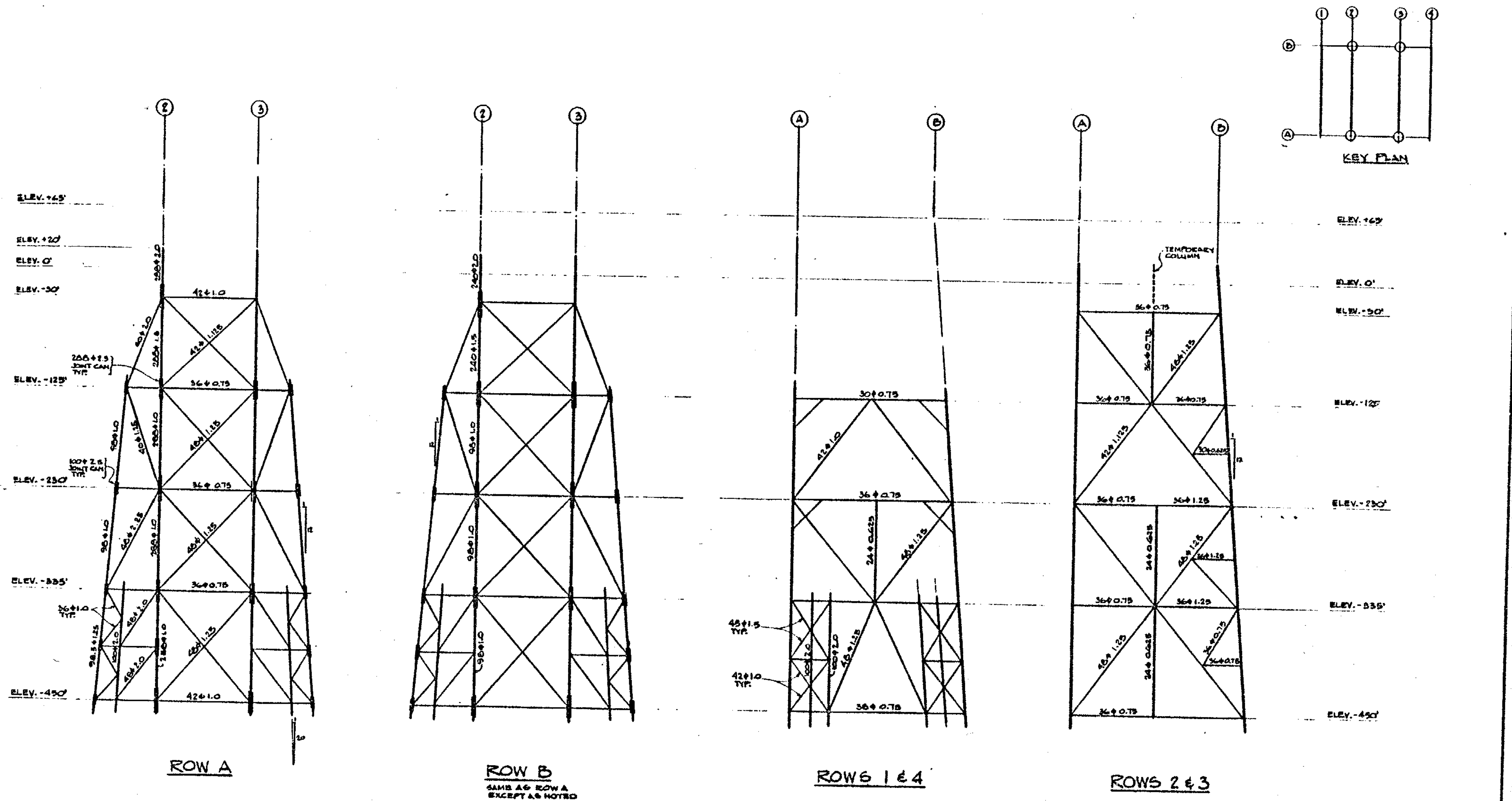
For

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

REVISION			Drawn	Scale
No	Date	Description	OWP	
1			Checked BMW	Date
2				
3			Approved	Job No.
4				
5			Approved	Client Approv
6				

Drawing Title: JACKET ELEVATIONS
NORTH ALEUTIAN SHELF
300,000 BYD CASE

Drawing No. 105 — Rev.



PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

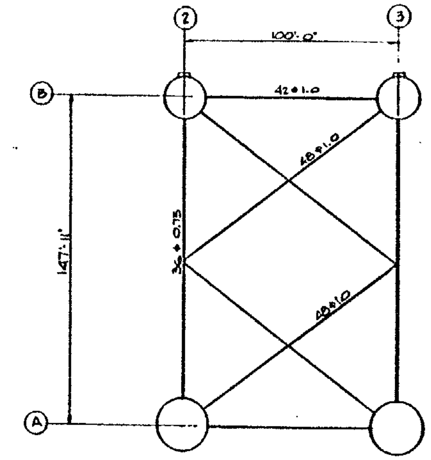
For

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

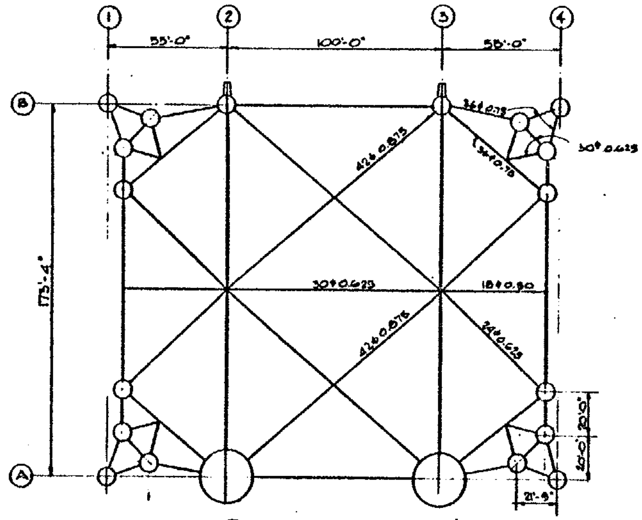
Drawing Title: JACKET ELEVATIONS NAVARIN BASIN 300 000 B/D CASE

Drawing No. 102

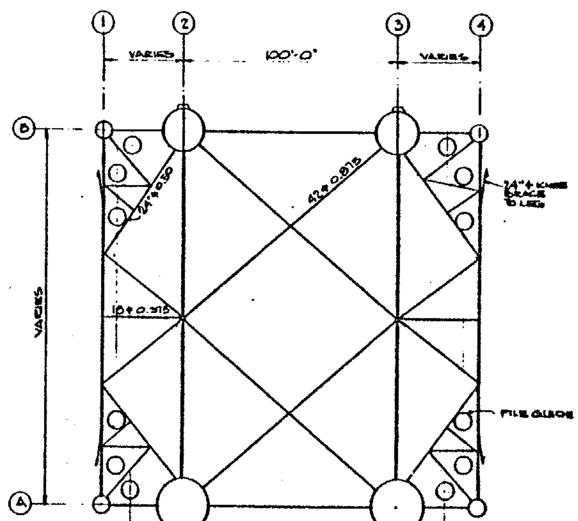
REVISION			Drawn	Scale
No	Date	Description	OWP	
1			Checked BMW	Date
2			Approved	Job No
3				
4				
5			Approved	Client Approv.
6				



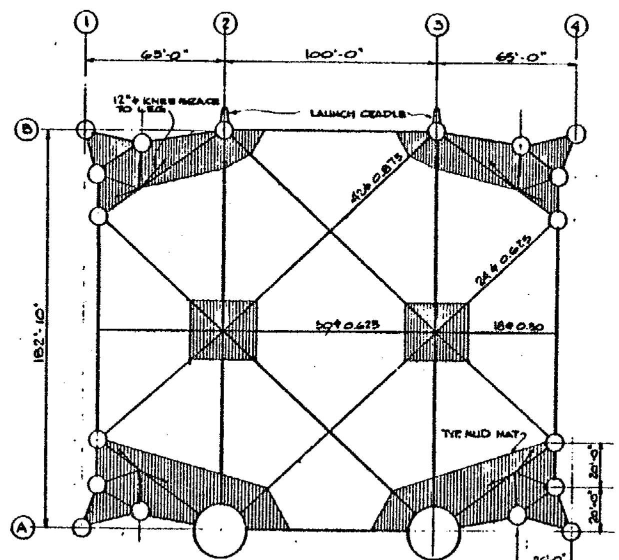
PLAN @ ELEV. -30'



PLAN @ ELEV. -335'



PLAN @ ELEVS. -125' & 230'



PLAN @ ELEV. -450'

REVISION				
No.	Date	Description		
1			Drawn OWT	Scale
2			Checked BMW	Date
3			Approved	Job No.
4				
5			Approved	Client Approv.
6				

PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

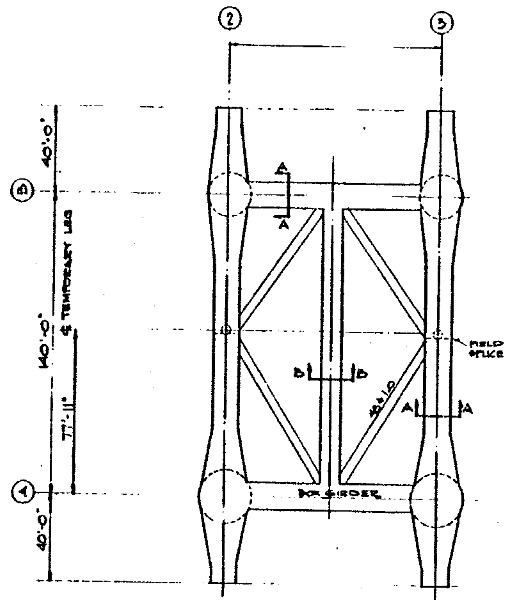
For

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

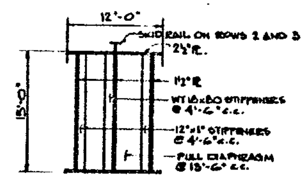
Drawing Title: STEEL TEMPLATE PLANS

Drawing No. 103

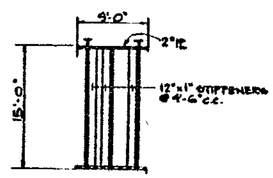
Rev.



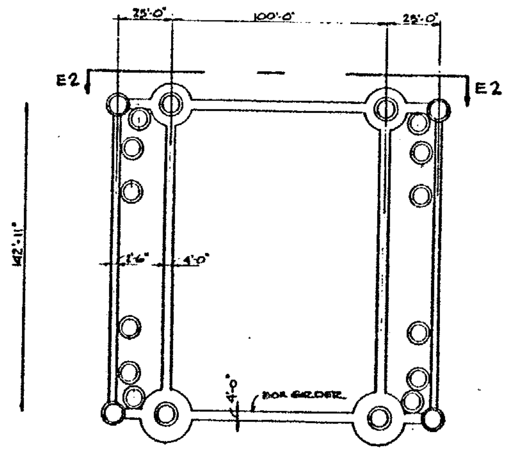
DECK SUPPORT FRAME
1" = 30'



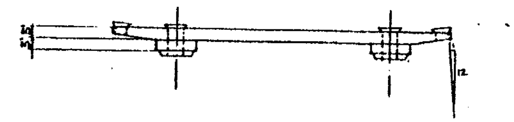
SECTION A



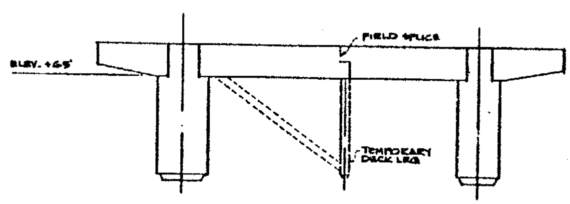
SECTION B
(SAME AS SECT. A. EXCEPT AS NOTED)



TEMPORARY PILE GUIDE DECK



ELEVATION E2



ELEVATION E1

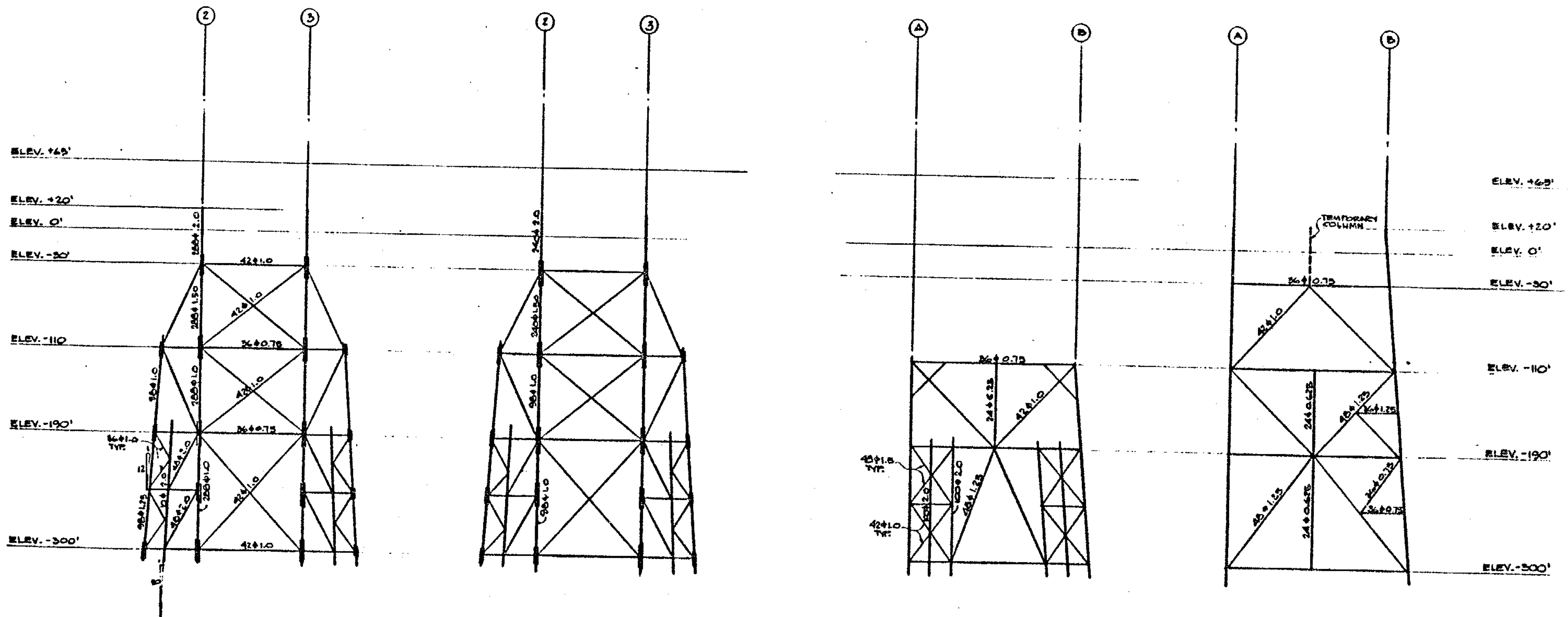
REVISION			Drawn	Scale
No	Date	Description	OWP	
1			Checked BMW	Date
2				
3			Approved	Job No.
4				
5			Approved	Client Approv.
6				

PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

For

SOUTHERN BERING SEA
PRODUCTION SYSTEMS STUDY

Drawing Title: DECK SUPPORT FRAME & PILE GUIDE DECK
Drawing No.: 104
Rev.



ROW A

ROW B
SAME AS ROW A
EXCEPT AS NOTED

ROWS 1 & 4

ROWS 2 & 3

REVISION			Drawn	Scale
No	Date	Description	DWF	
1			Checked BMW	Date
2				
3			Approved	Job No.
4				
5			Approved	Client Approv
6				

PMB SYSTEMS ENGINEERING INC.
Offshore Engineering Division
San Francisco Houston

For

SOUTHERN BERING SEA
PRODUCTION SYSTEMS STUDY

Drawing Title
JACKET ELEVATIONS
NORTH ALEUTIAN SHELF
300,000 B/D CASE

Drawing No.
105

Rev

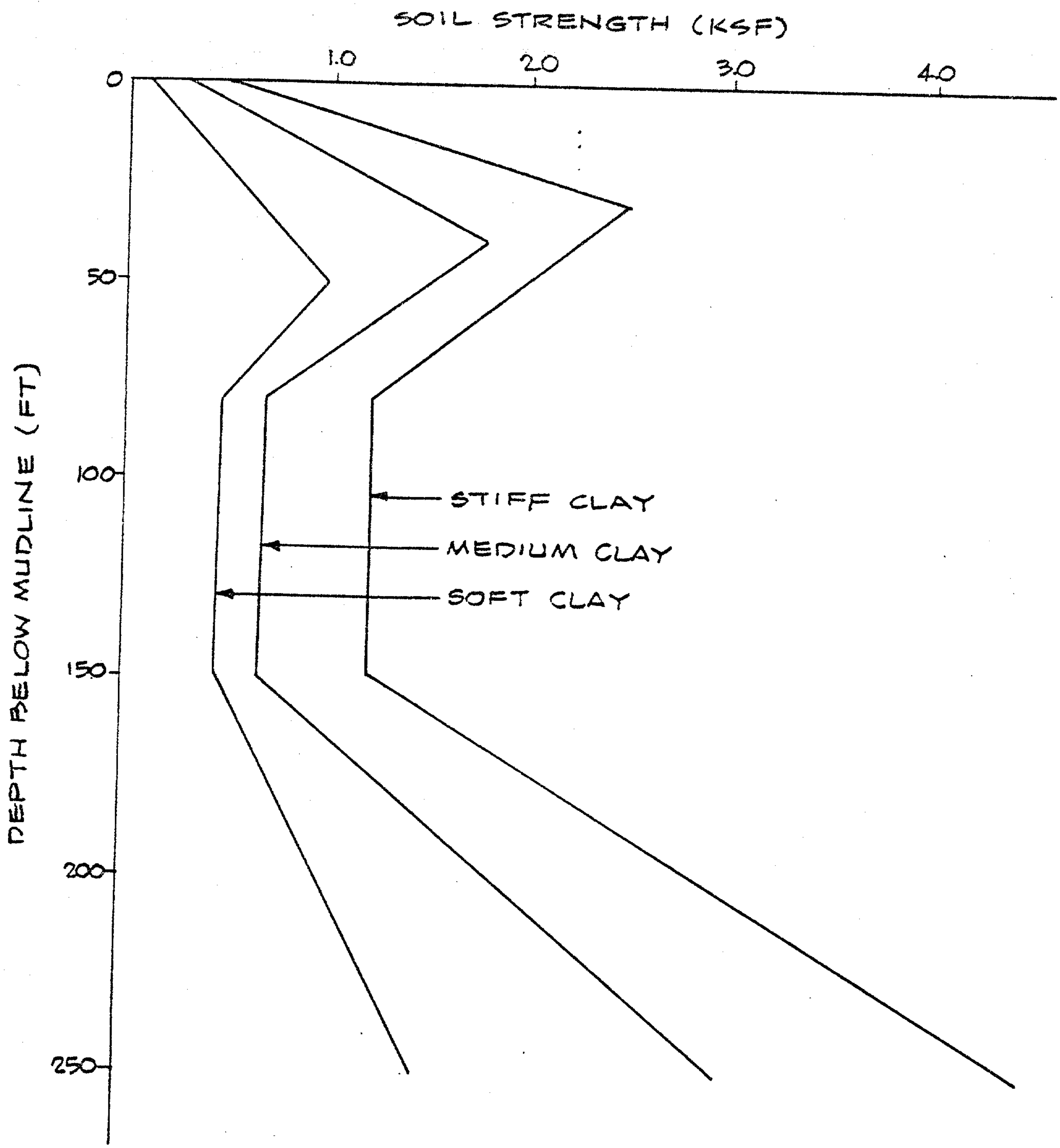


FIGURE 4-1 SOIL TYPE A, COHESIVE - ASSUMED SOIL STRENGTH FOR ST. GEORGE AND NAVARIN BASINS

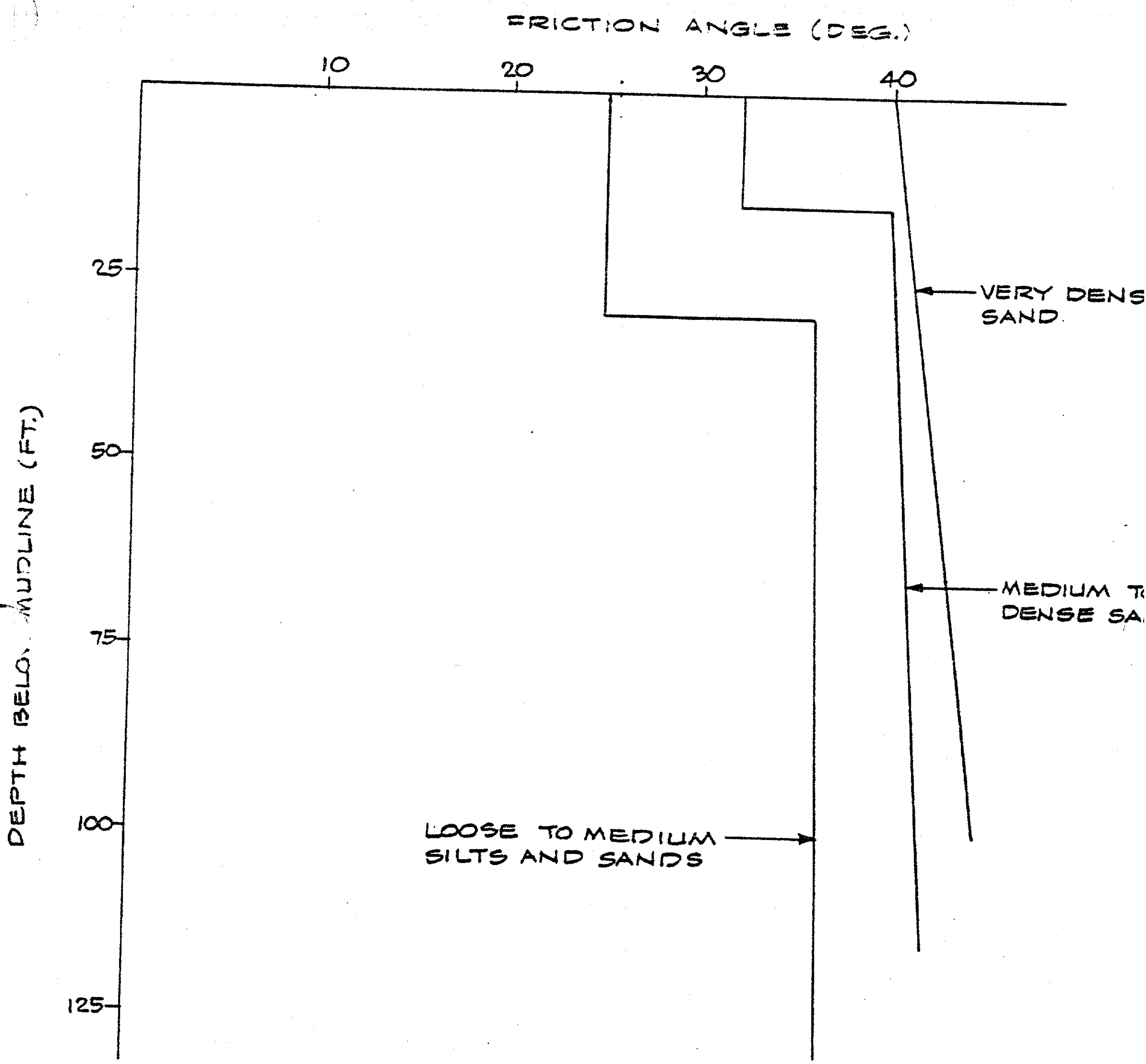
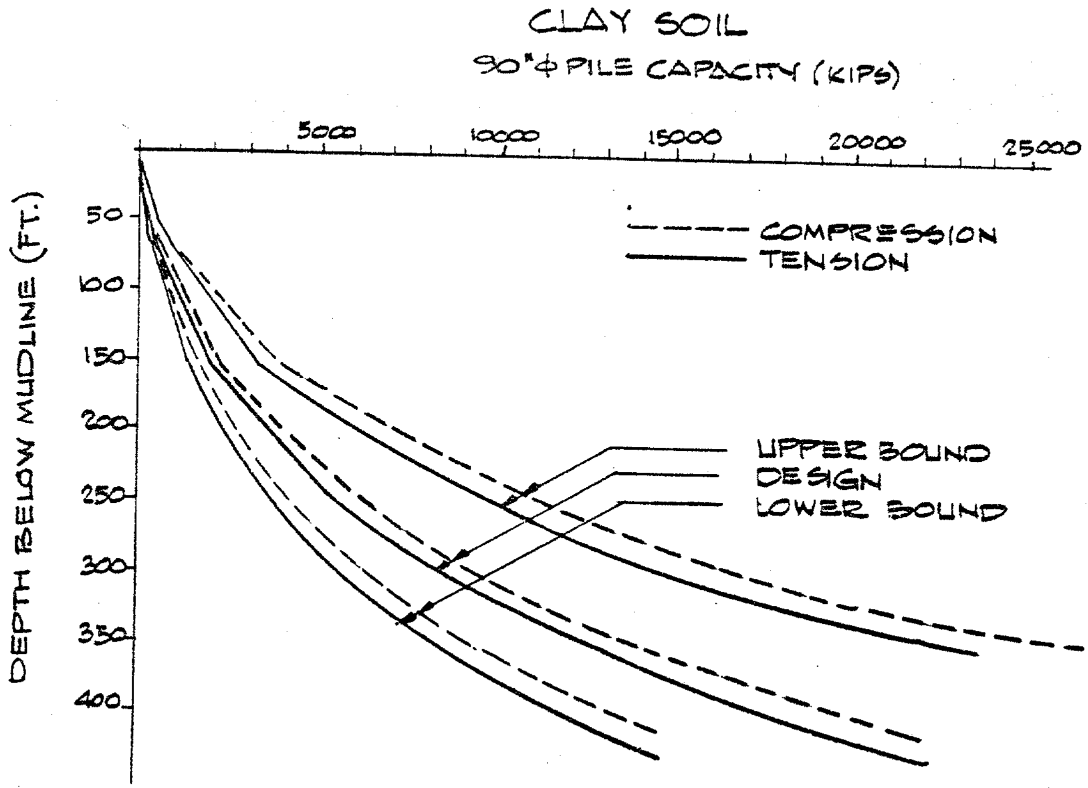
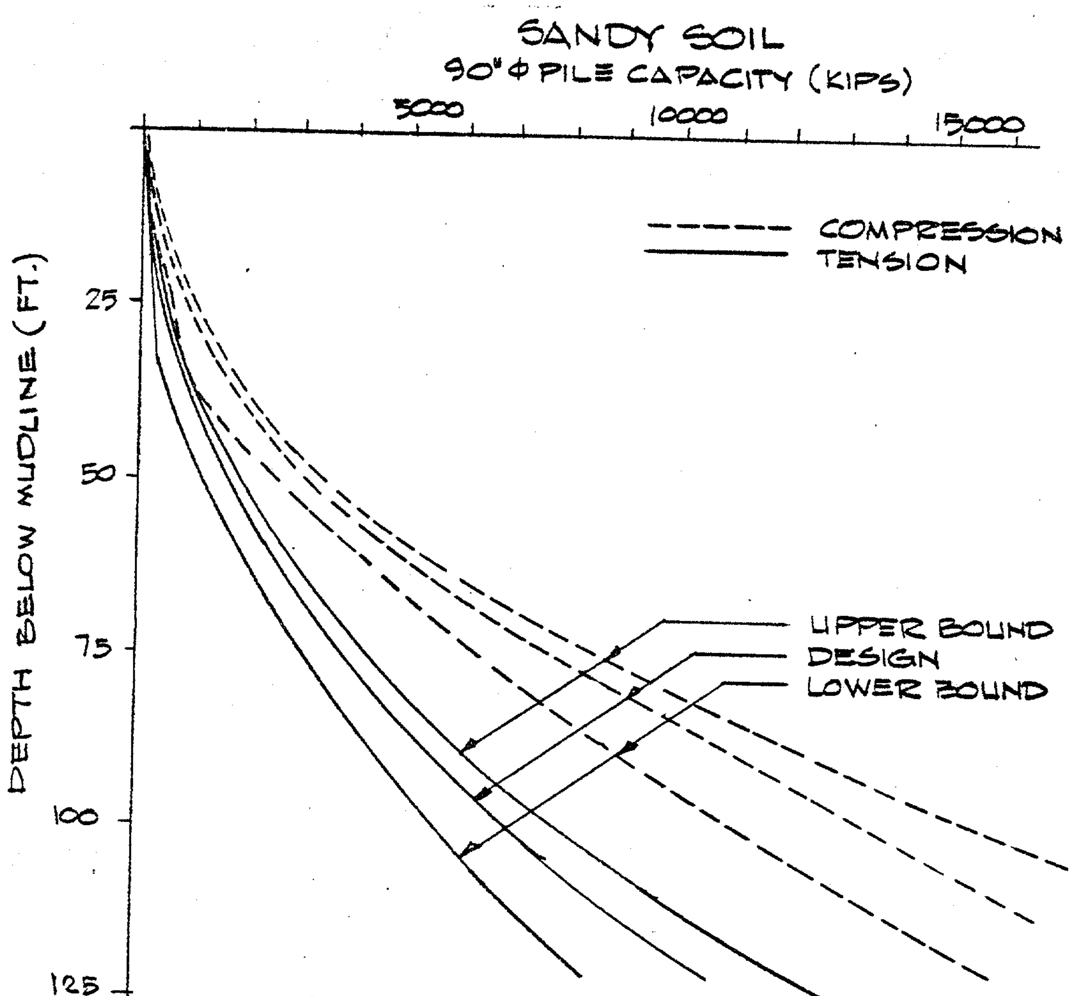


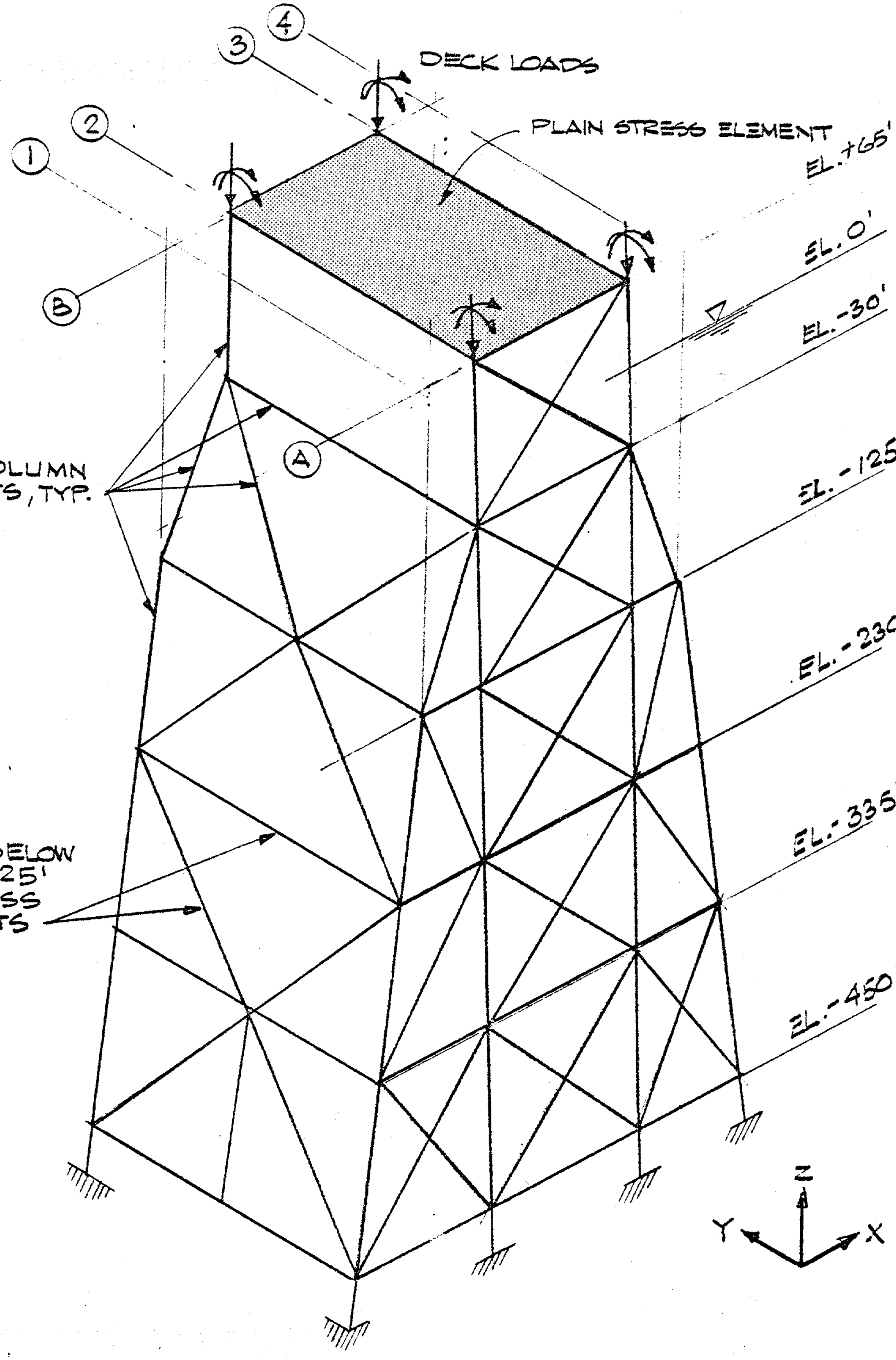
FIGURE 4-2 SOIL TYPE B, GRANULAR - ASSUMED SOIL PROPERTIES FOR ST. GEORGE BASIN, NAVARIN BASIN AND THE NORTH ALEUTIAN SHELF





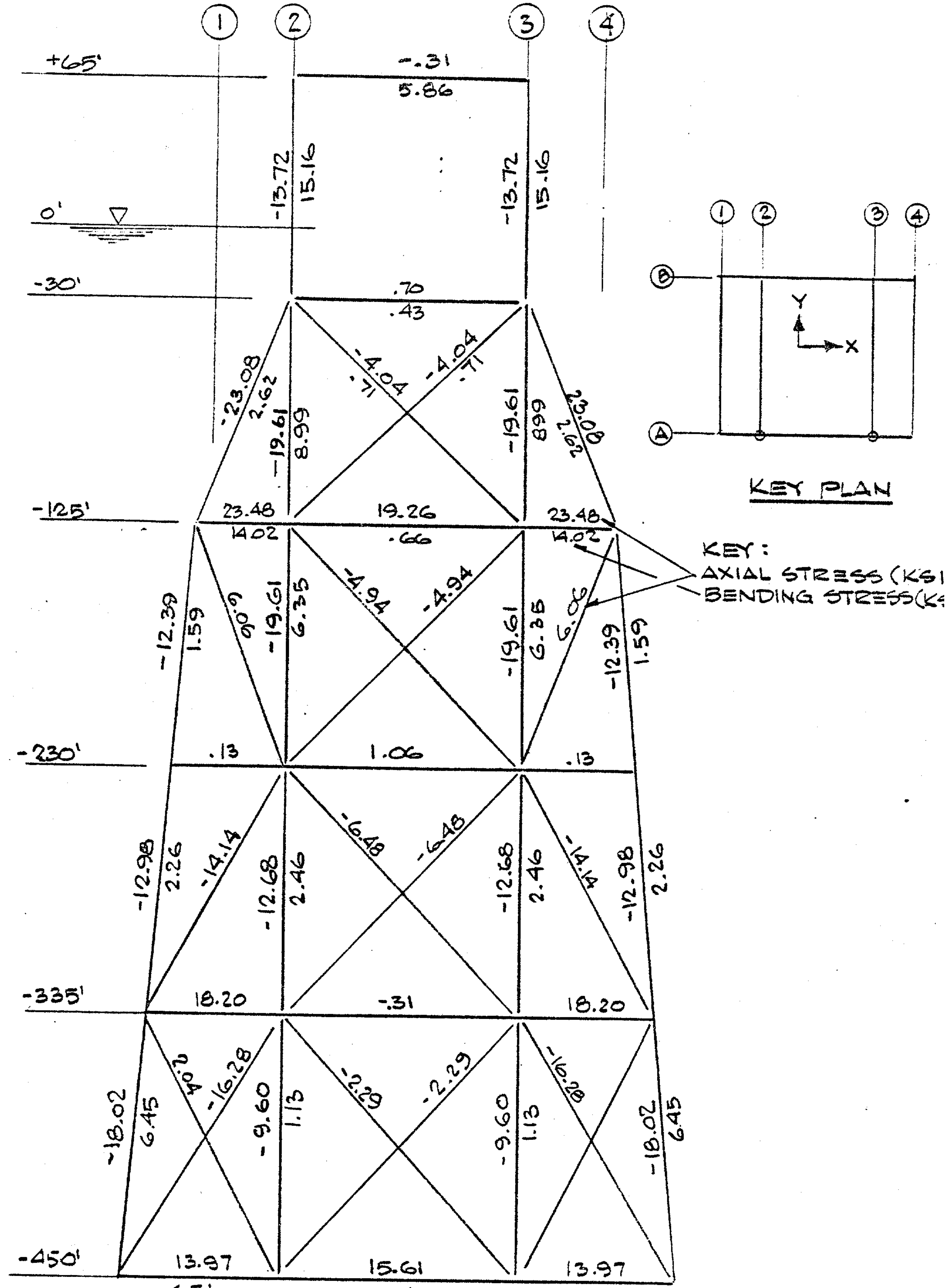
PILE AXIAL CAPACITIES

FIGURE 4-3



FINITE ELEMENT MODEL

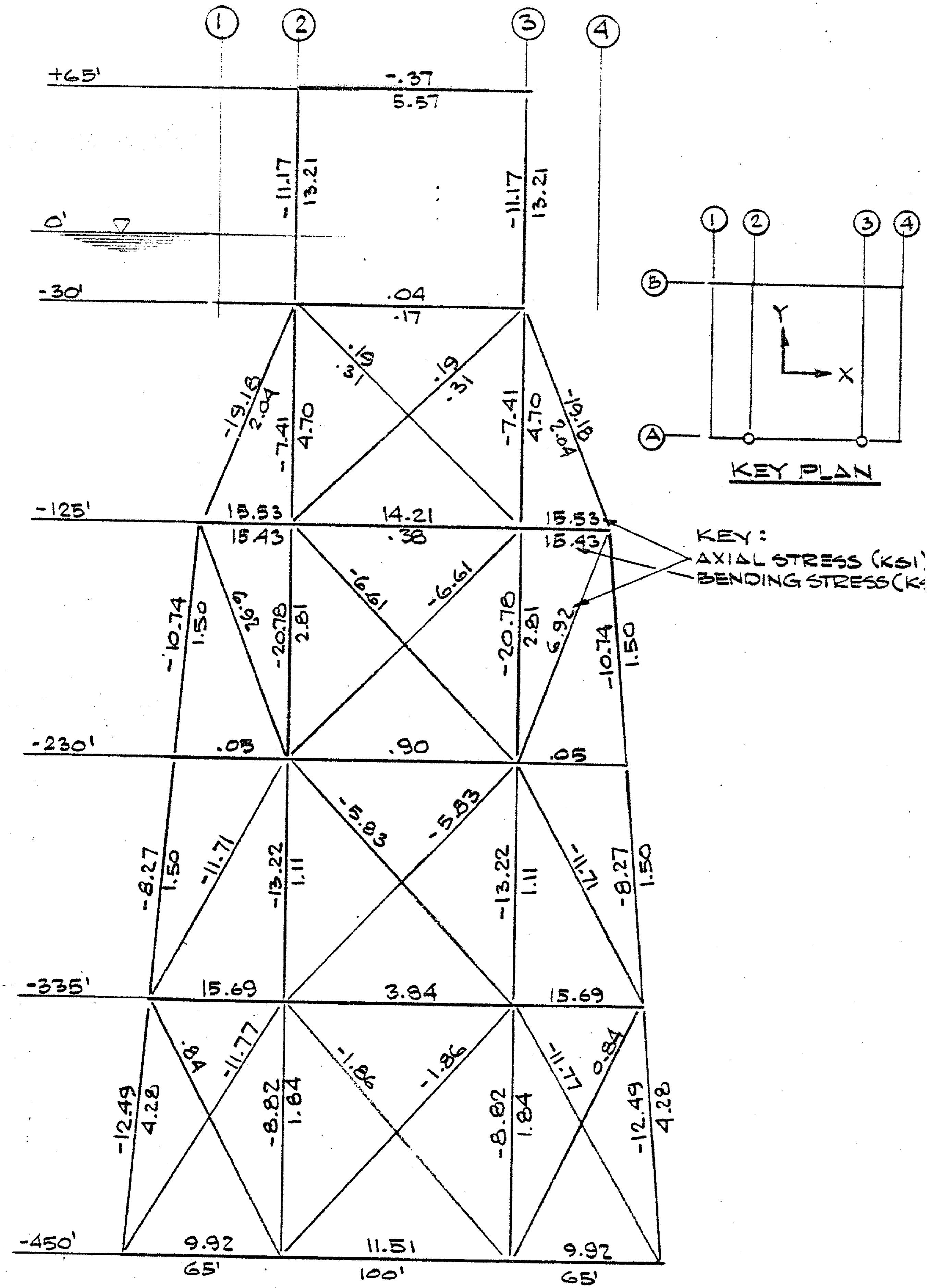
FIGURE 4-4



NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION I

(ST.GEORGE 300000 BPD)

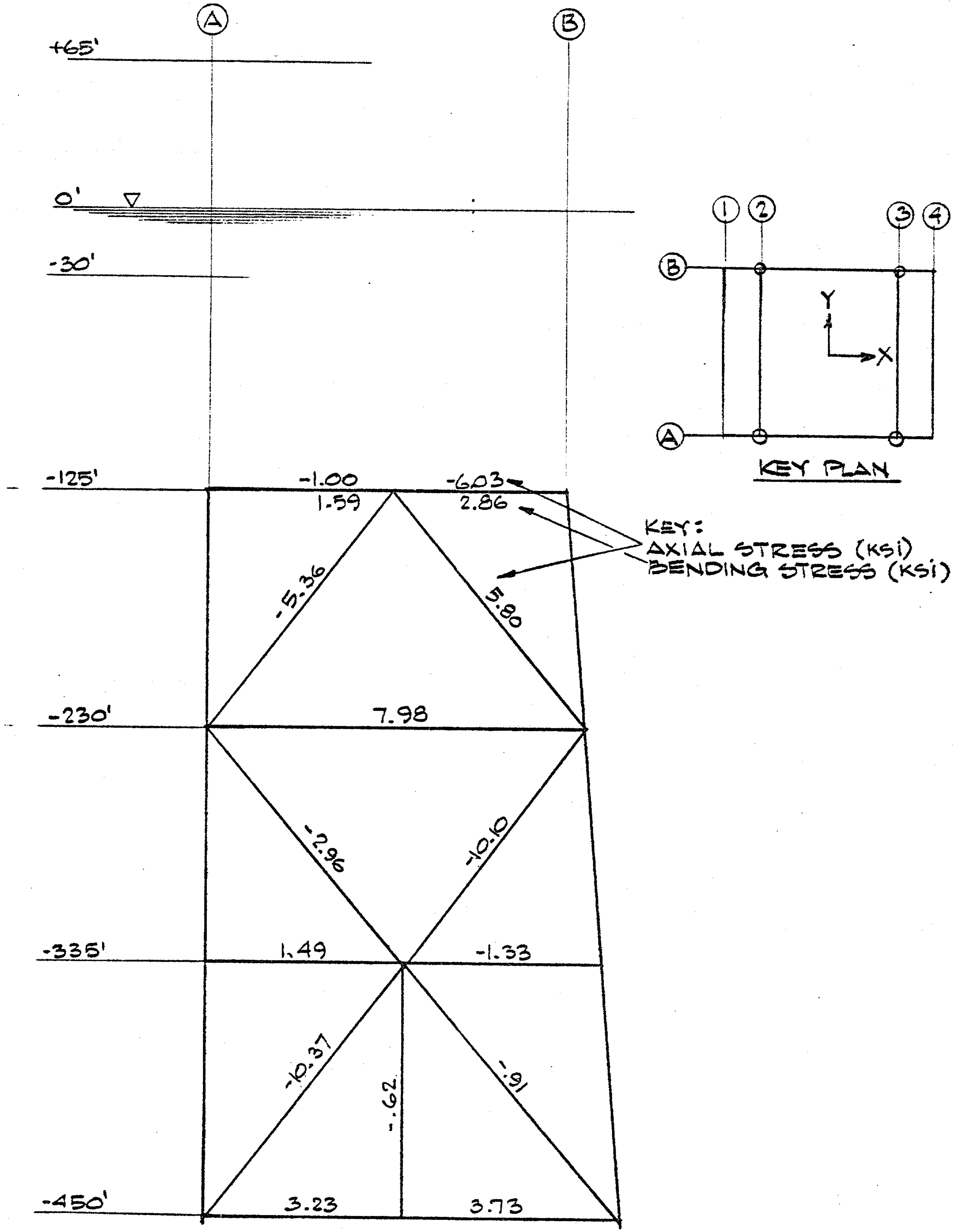
FIGURE 4-5



NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION I

(ST. GEORGE 300000 BPD)

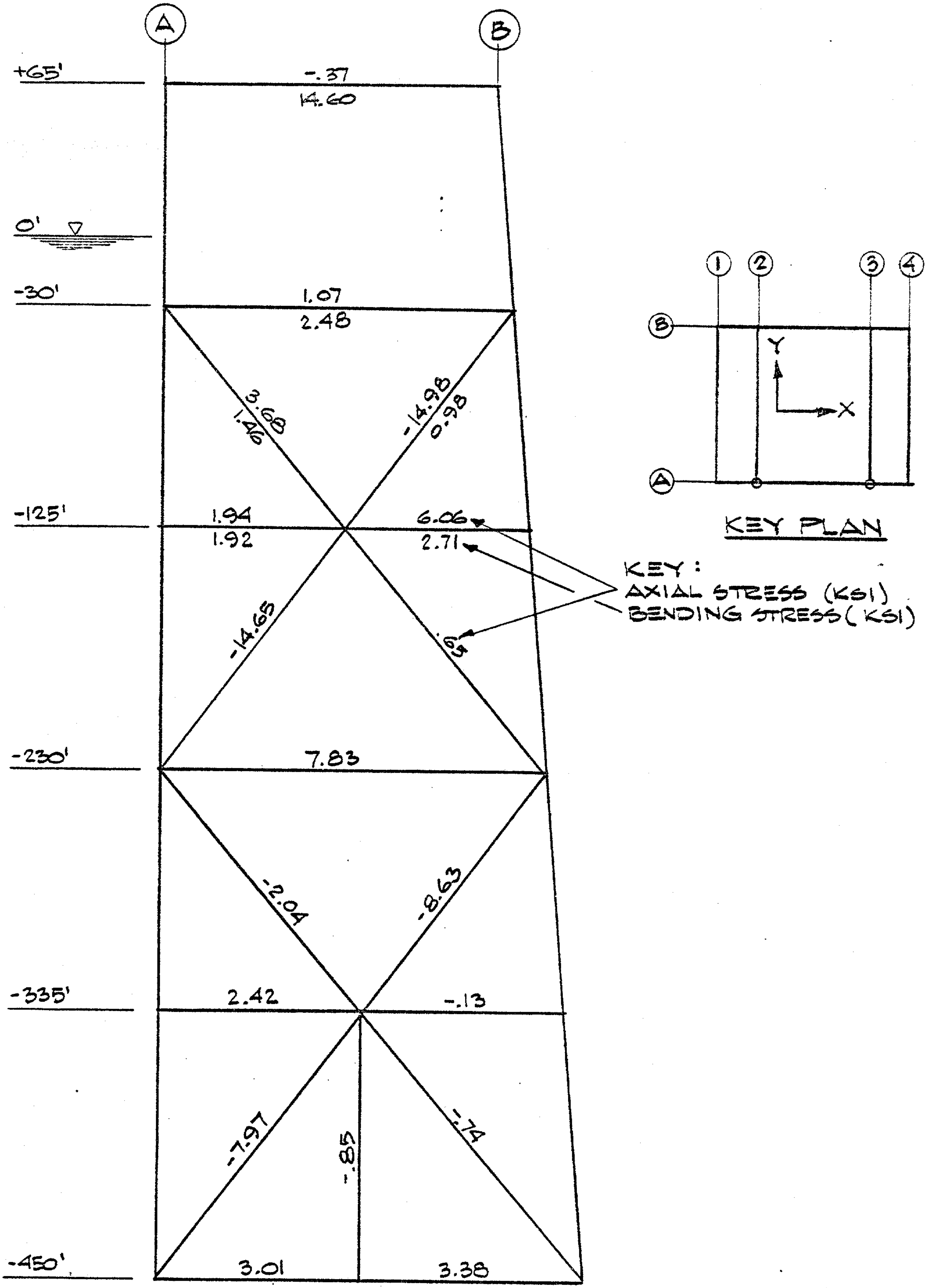
FIGURE 4-6



NOMINAL STRESSES ON ROWS 1 & 4 DUE TO LOAD COMBINATION

(ST. GEORGE 300 000 BPD)

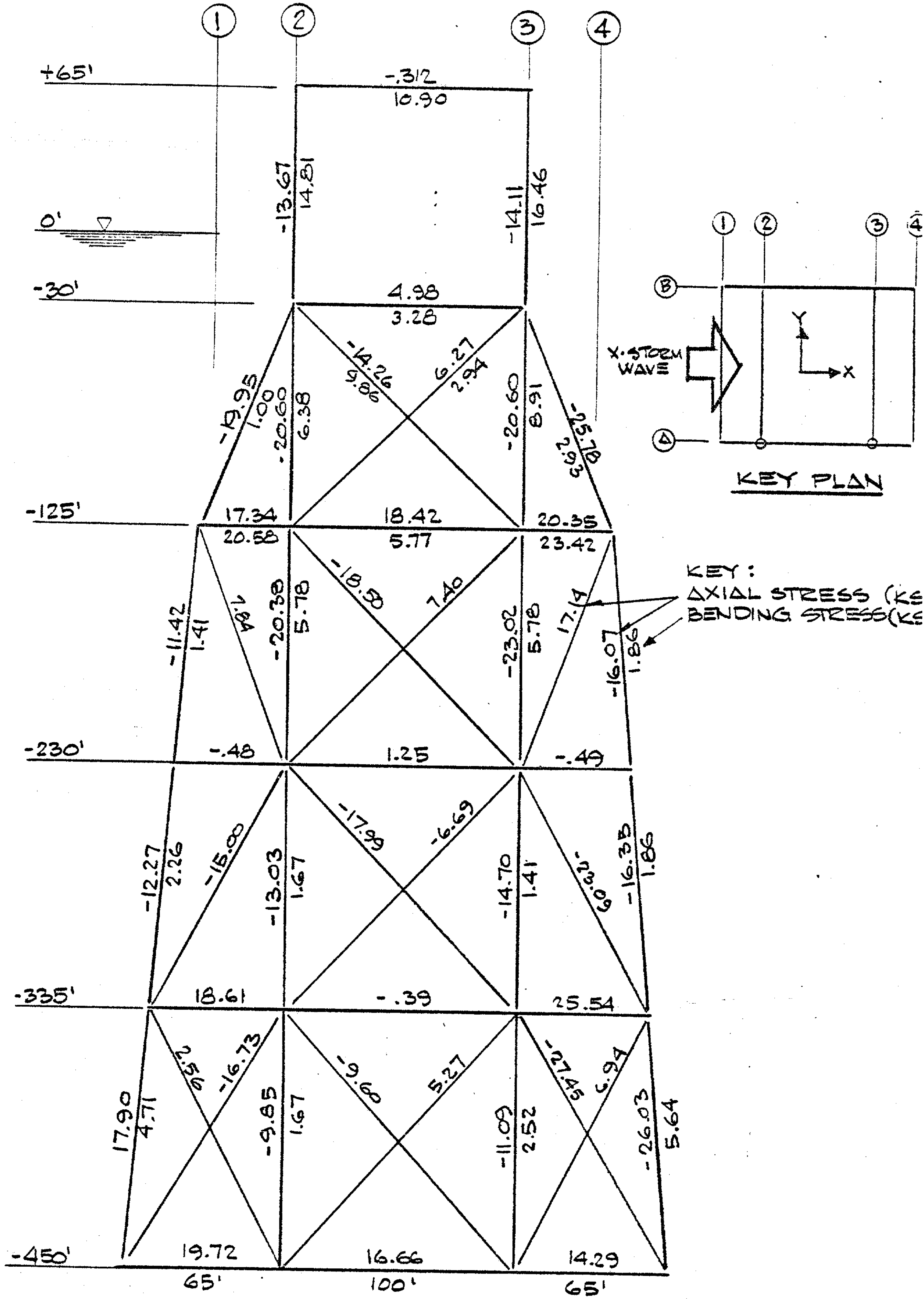
FIGURE 4-7



NOMINAL STRESSES ON ROWS 2 & 3 DUE TO LOAD COMBINATION

(ST. GEORGE, 300 000 BPD)

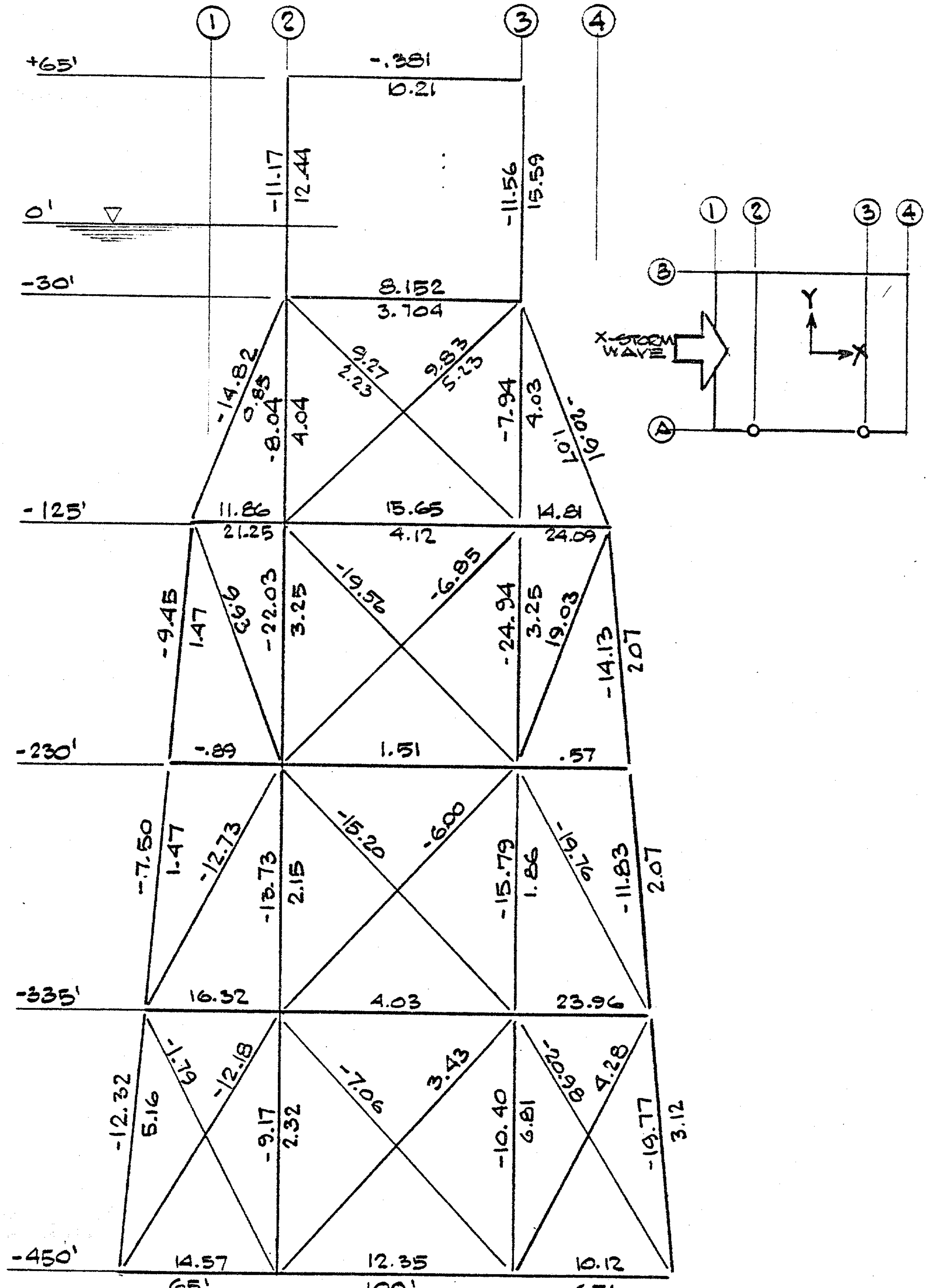
FIGURE 4-8



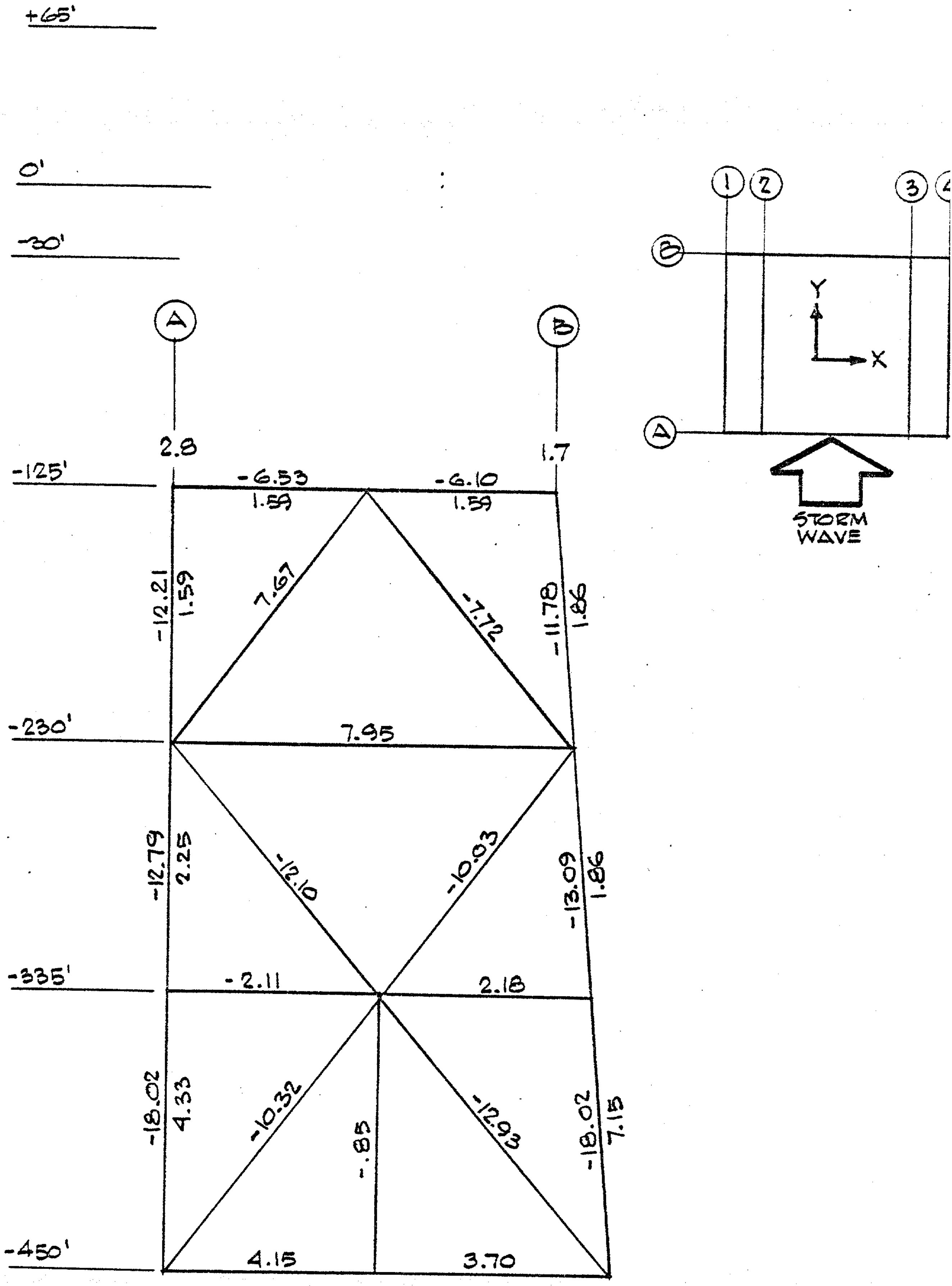
NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 2 WITH STORM IN X-DIRECTION

(ST. GEORGE 300000 BPD)

FIGURE 4.9

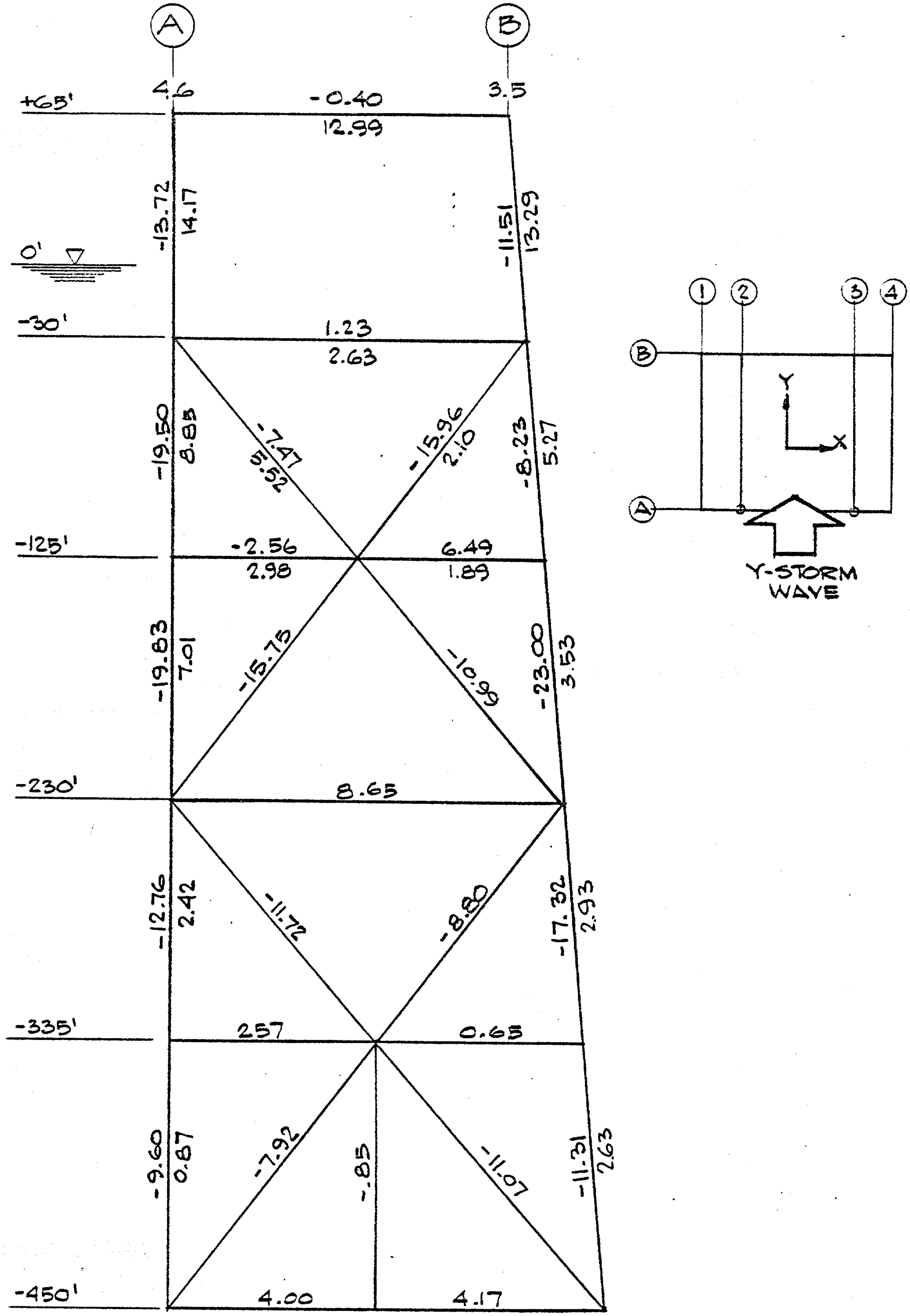


NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION 2
WITH STORM IN X-DIRECTION
(ST. GEORGE, 300000 BPD)



NOMINAL STRESSES ON ROWS 1 & 4 DUE TO LOAD COMBINATION
WITH STORM IN Y - DIRECTION
(ST. GEORGE 300 000 BPD)

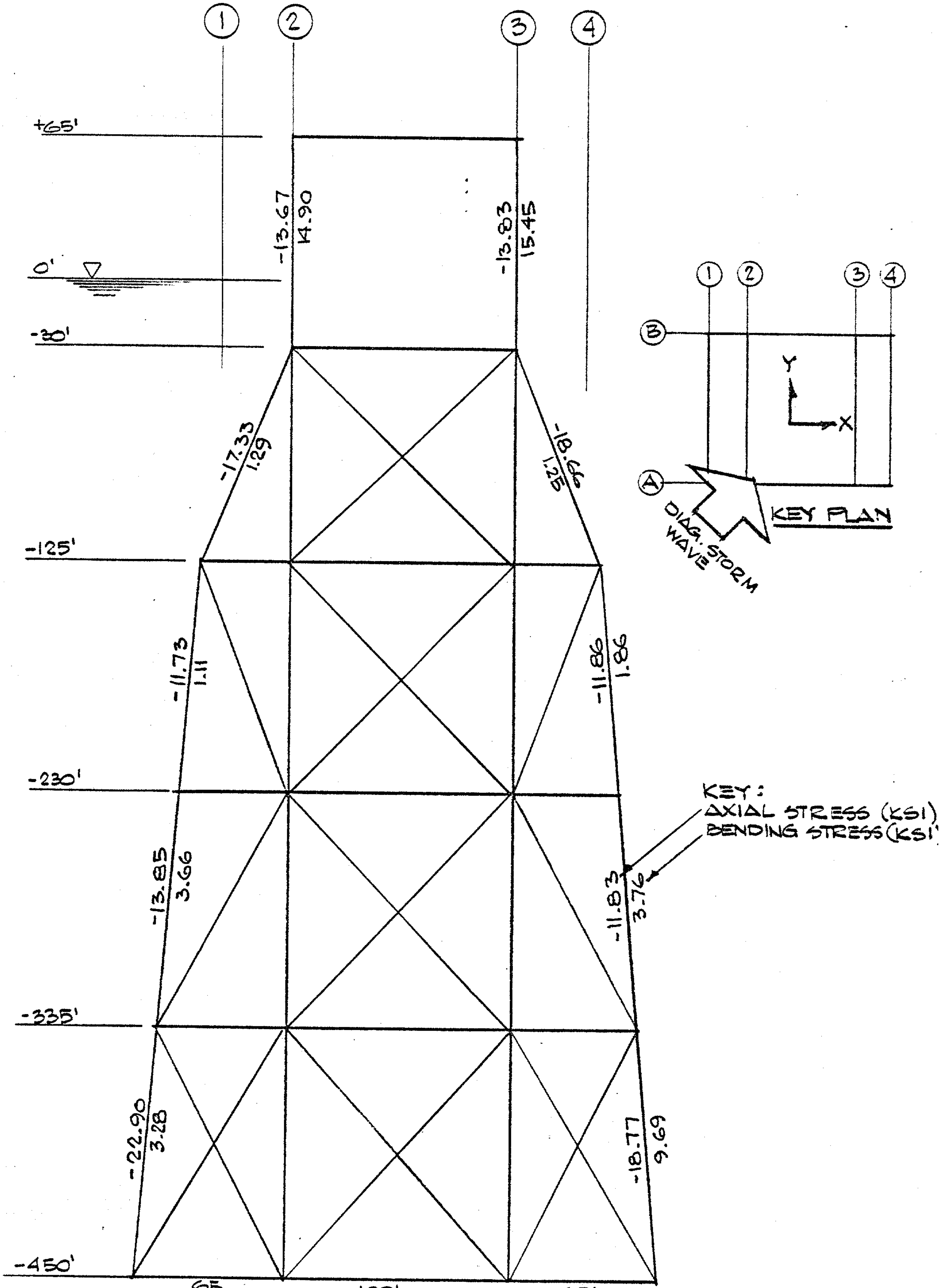
FIGURE 4.11



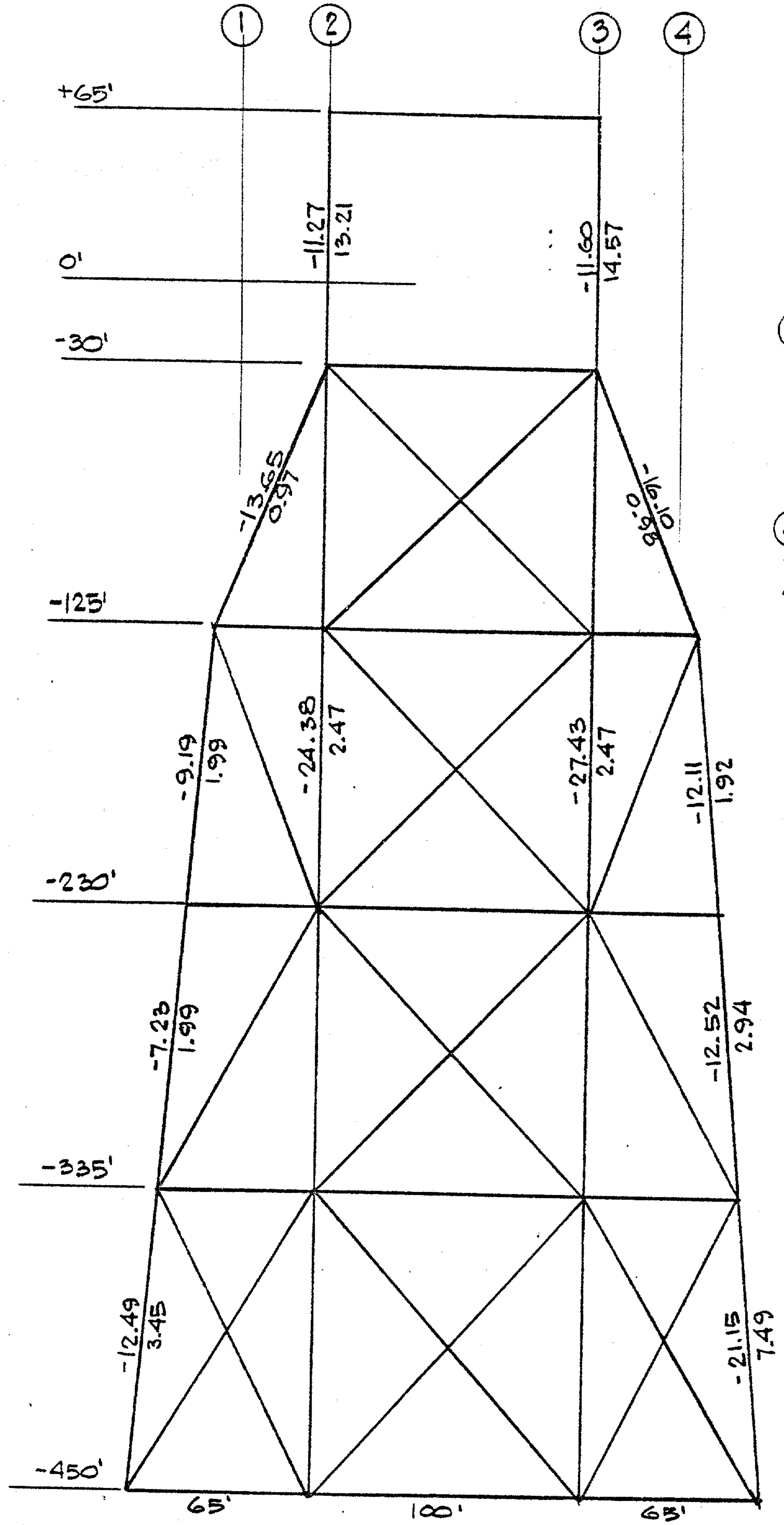
NOMINAL STRESSES ON ROW 2 & 3 DUE TO LOAD COMBINATION 2

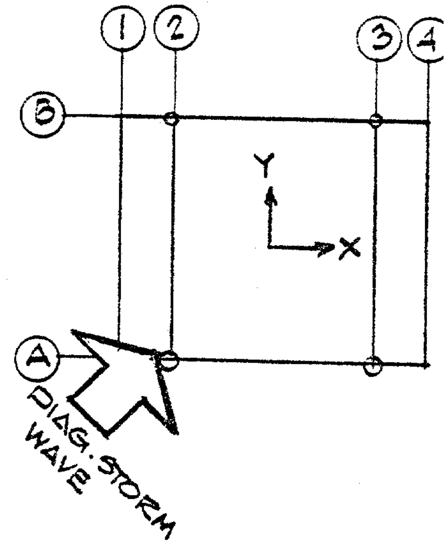
WITH STORM IN Y-DIRECTION

(ST. GEORGE 300000 BPD)



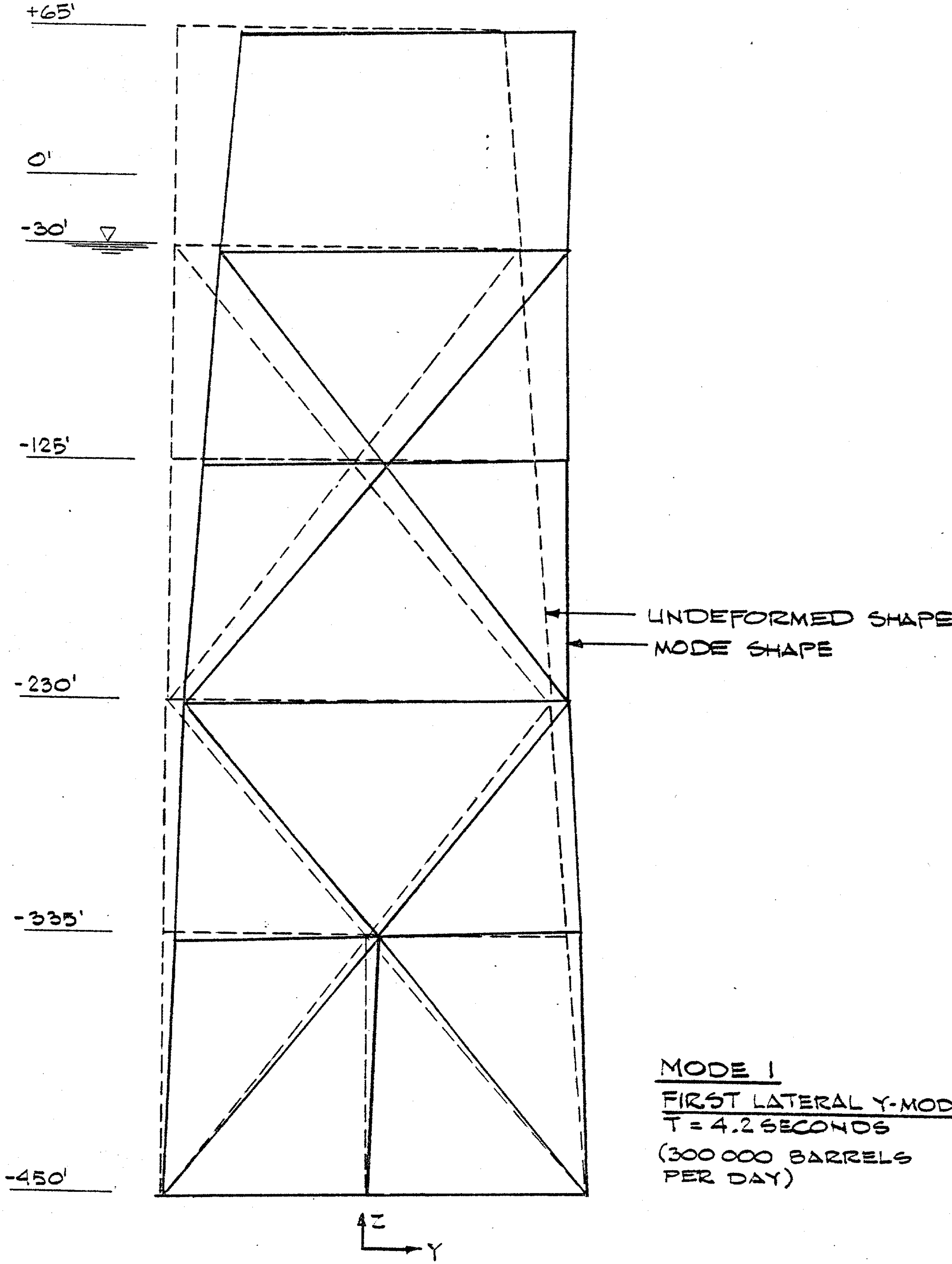
NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 2 WITH STORM DIAGONALLY (ST. GEORGE 300000 BPD)





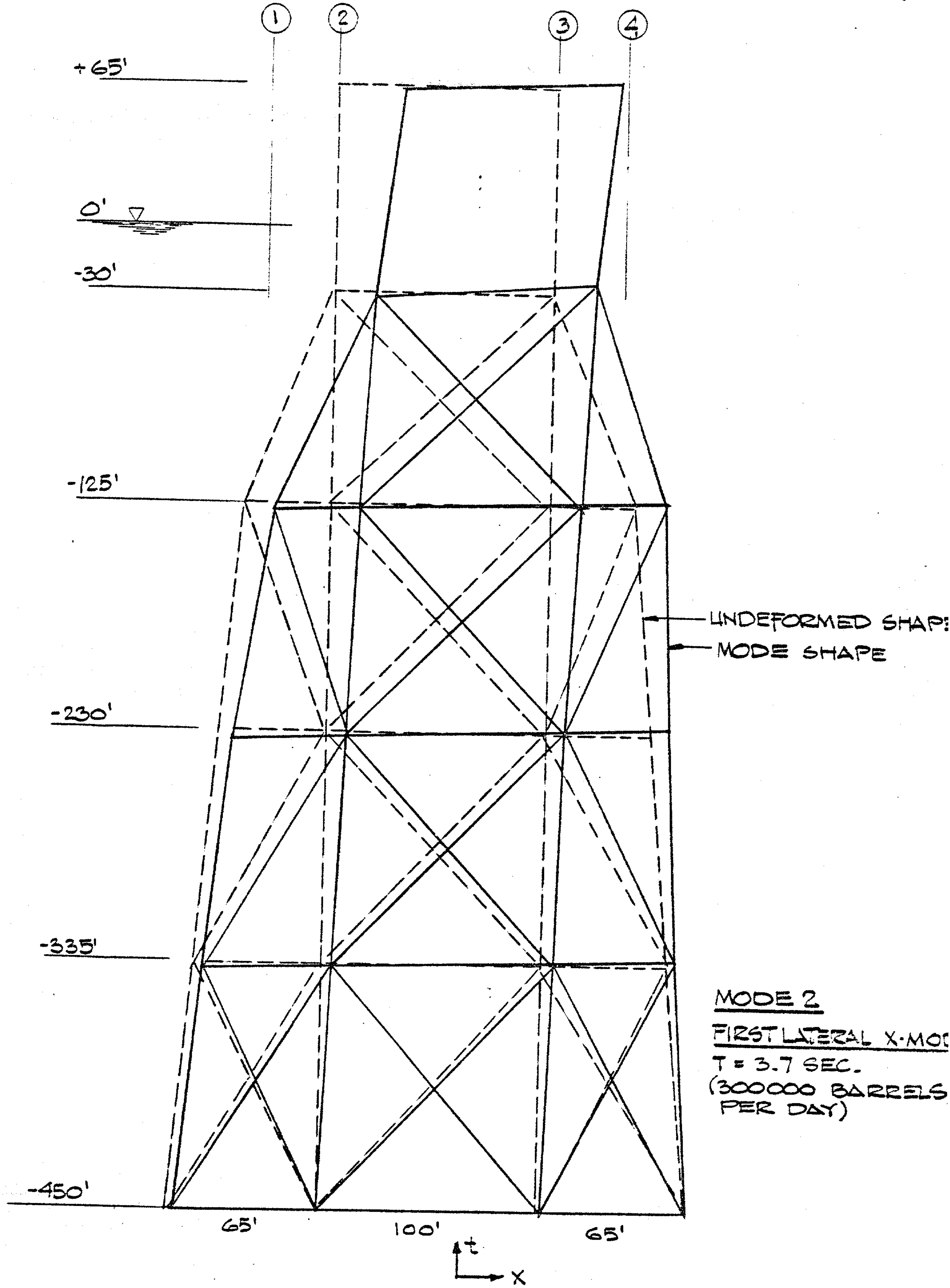
NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION 2
WITH STORM DIAGONALLY
(ST. GEORGE, 300000 BPD)

FIGURE 4-14



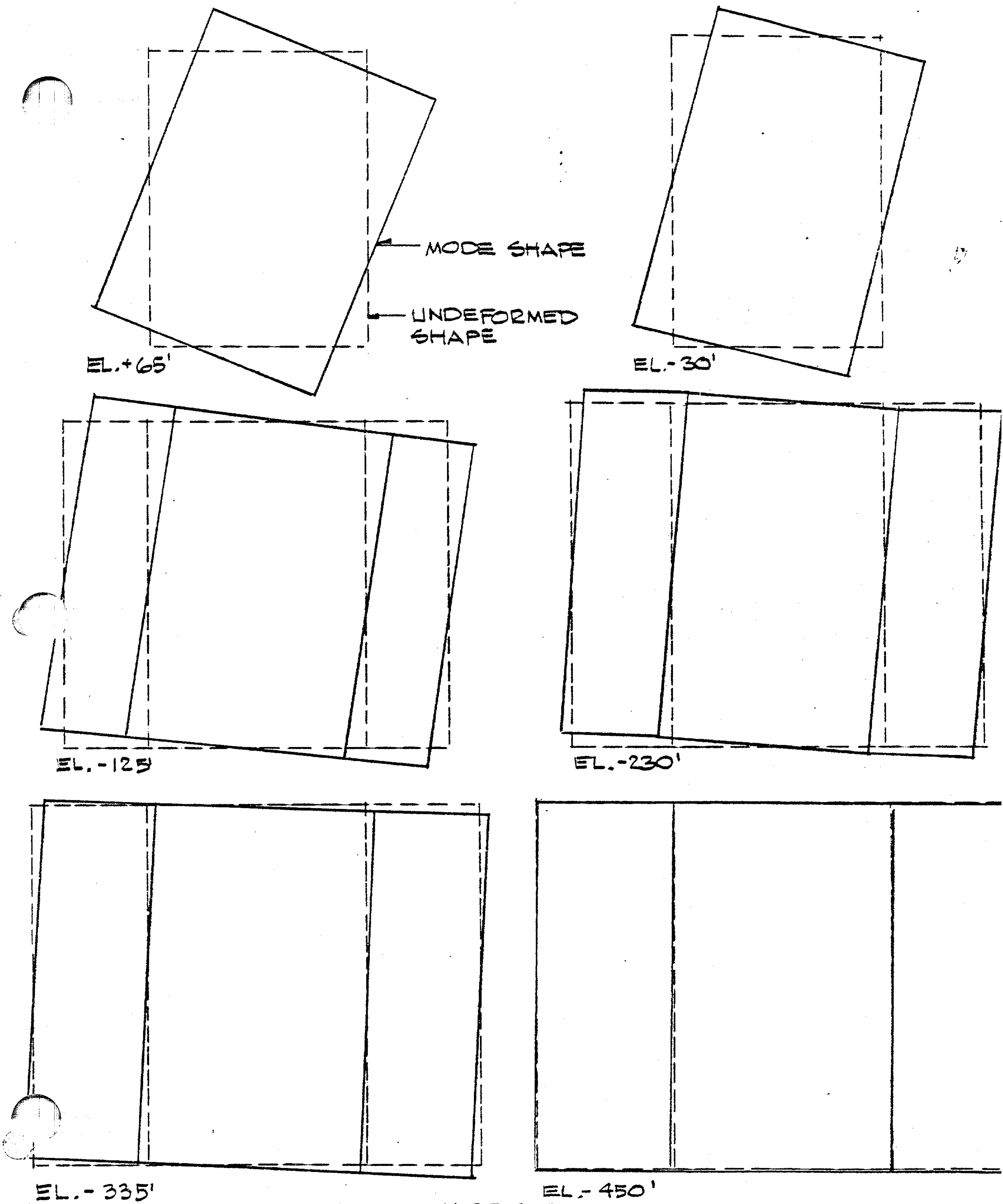
MODE 1
FIRST LATERAL Y-MODE
T = 4.2 SECONDS
(300 000 BARRELS PER DAY)

ROWS 2 OR 3
FIGURE 4-15



ROW A

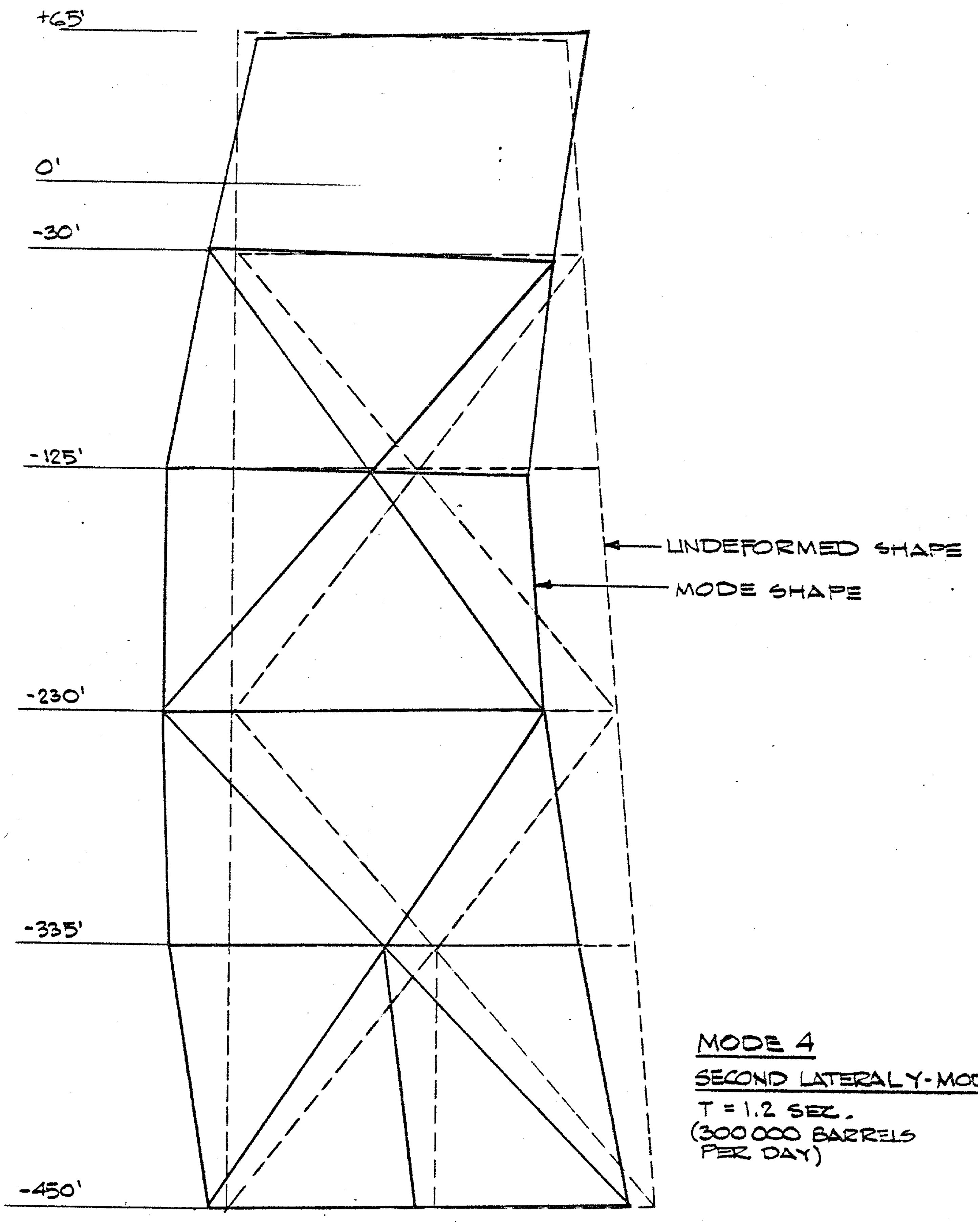
FIGURE 4-16



MODE 3

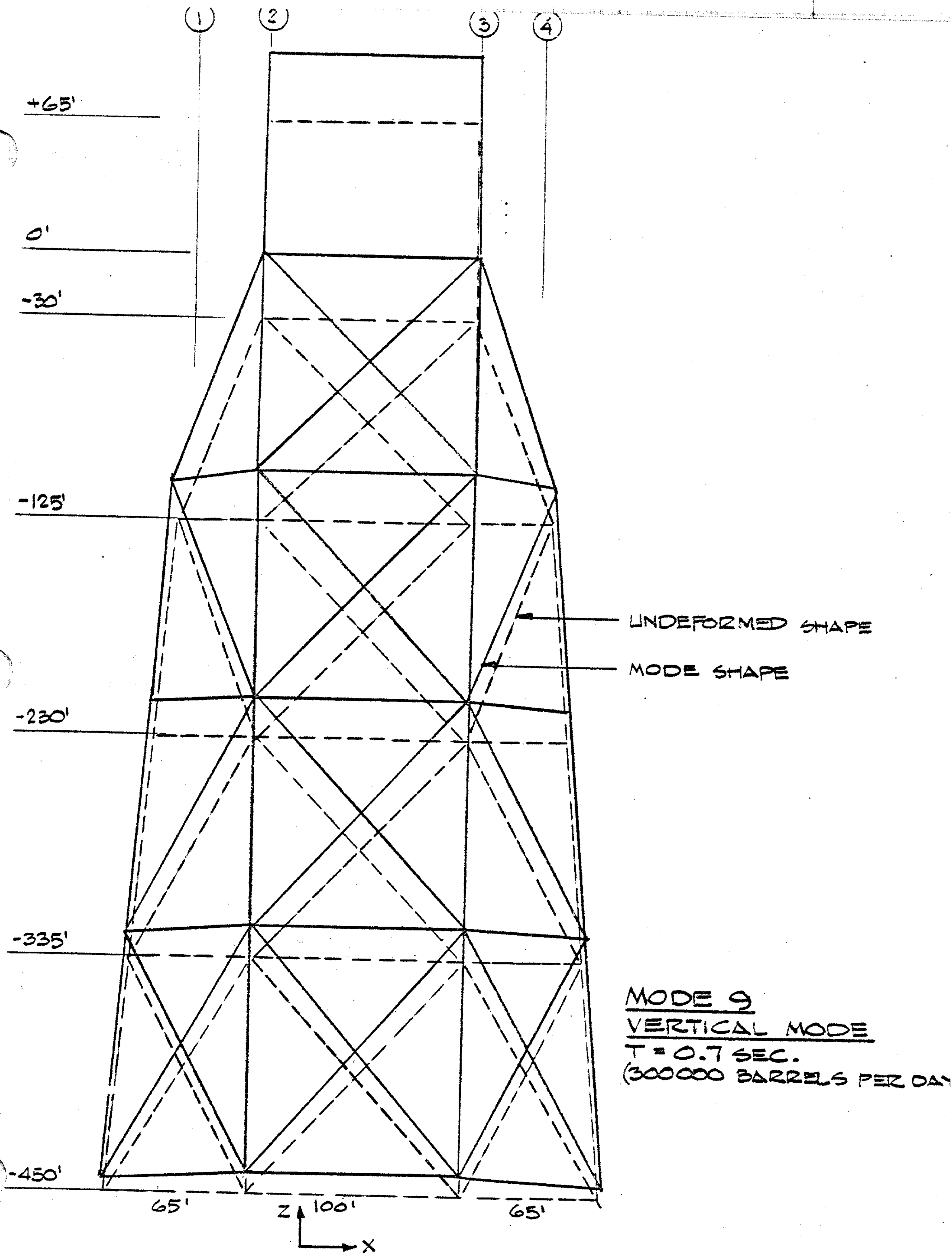
FIRST TORSIONAL MODE

T = 2.8 SEC. (300000 BARRELS PER DAY)



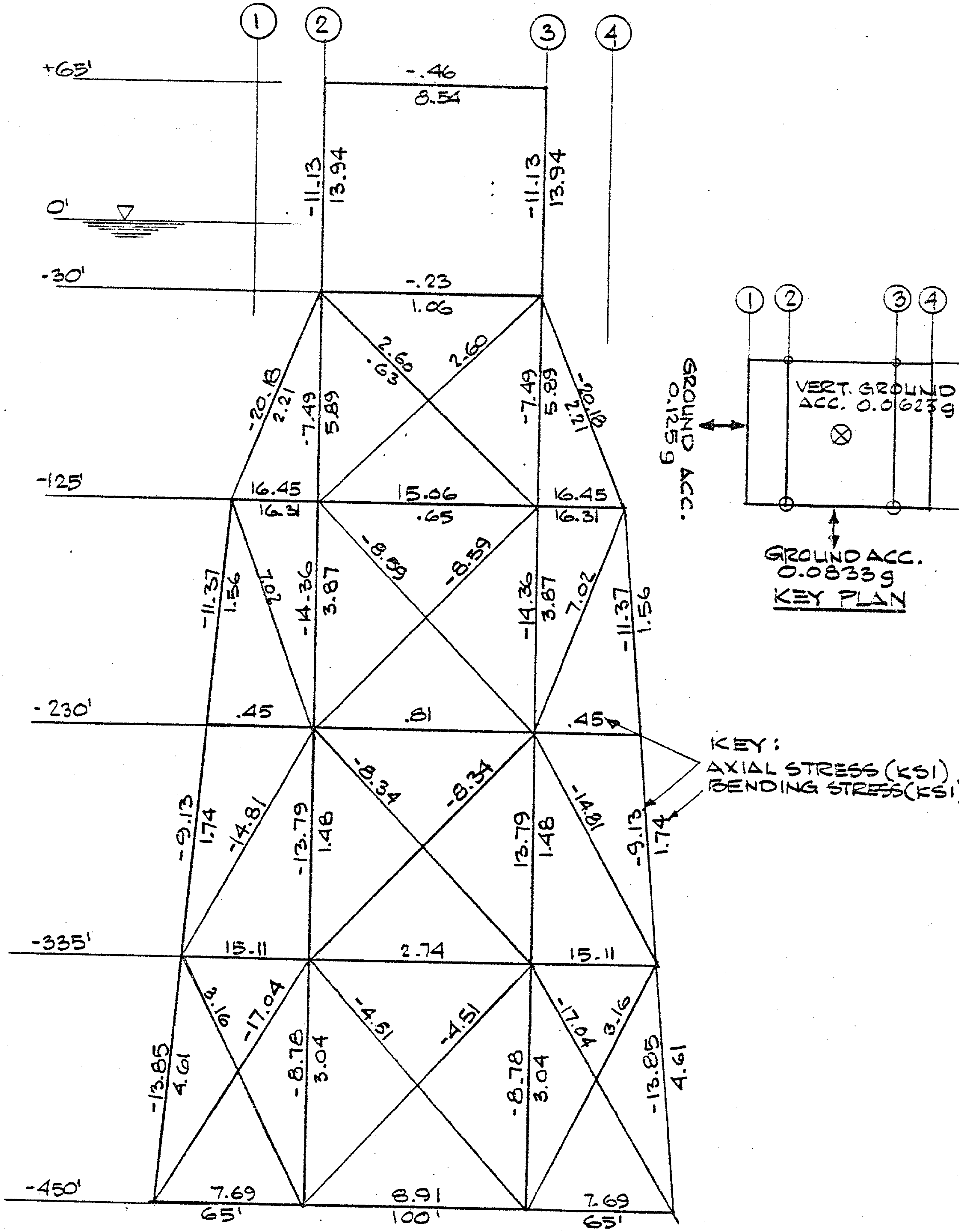
ROWS 2 & 3

FIGURE 4-18



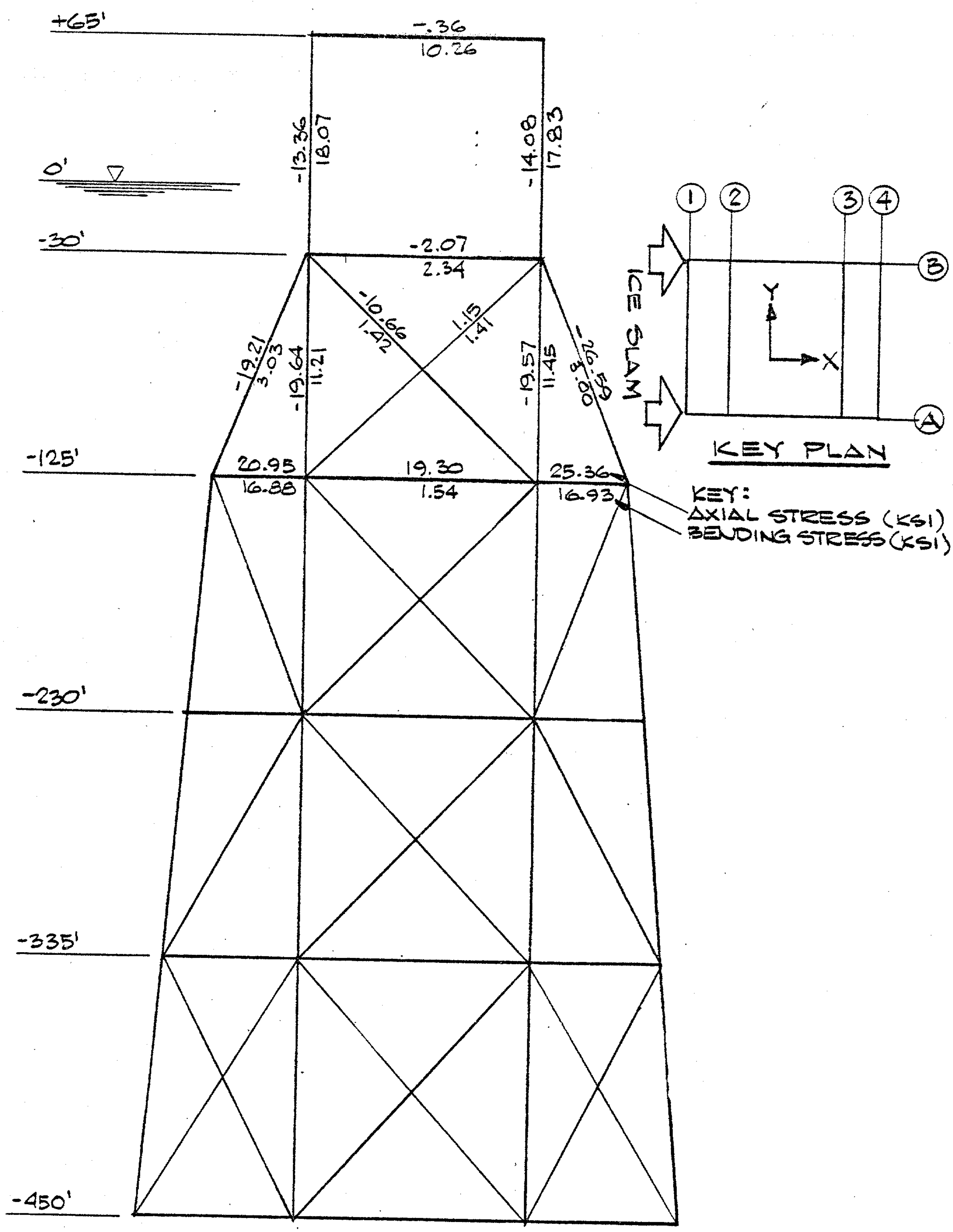
ROW A

FIGURE 4-19



NOMINAL STRESSES ON ROW B DUE TO STRENGTH SEISMIC LOADING - LOAD COMBINATION 3

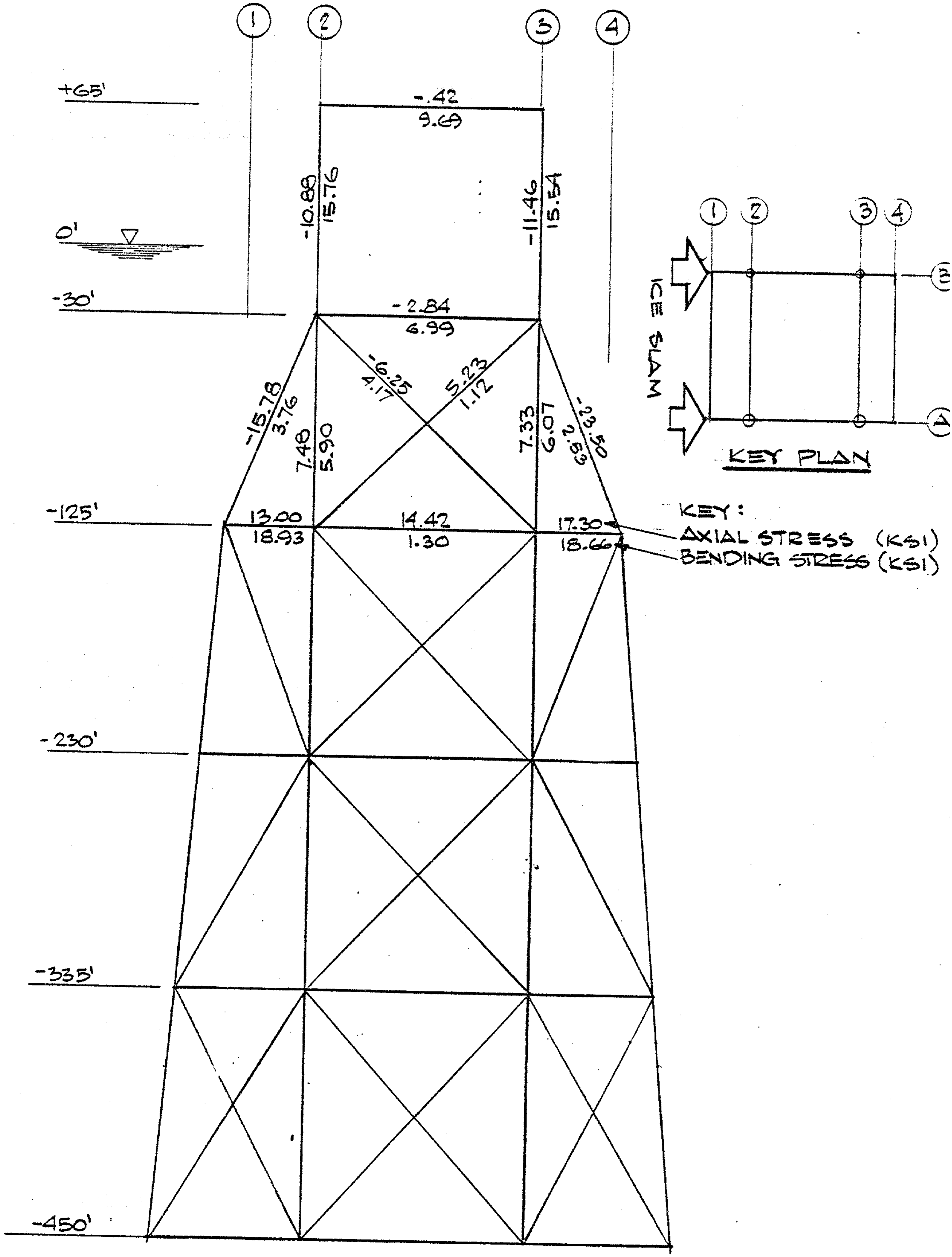
(ST. GEORGE 300 000 BPD)



NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 5

(ST. GEORGE 300000 BPD)

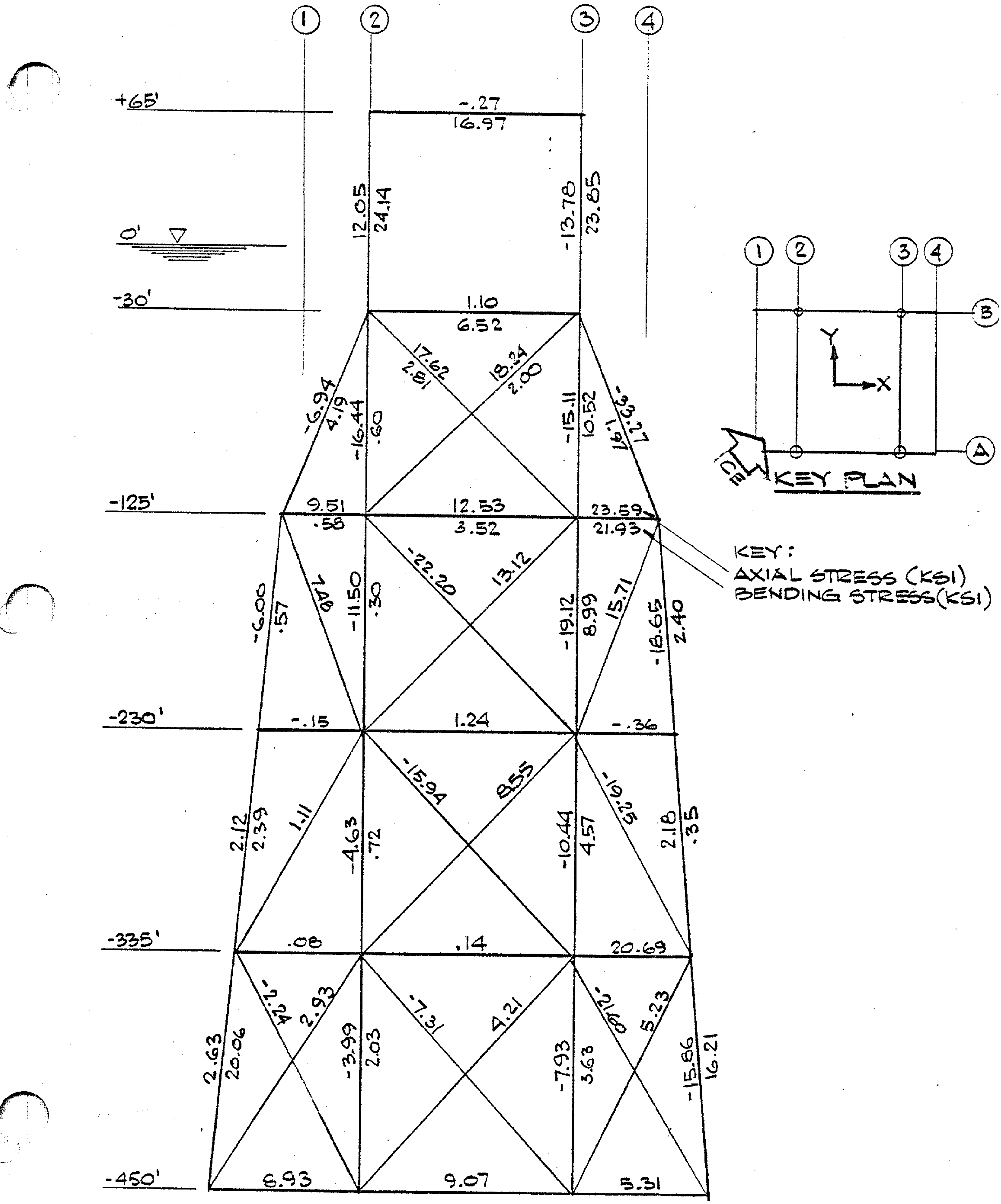
FIGURE 4-21



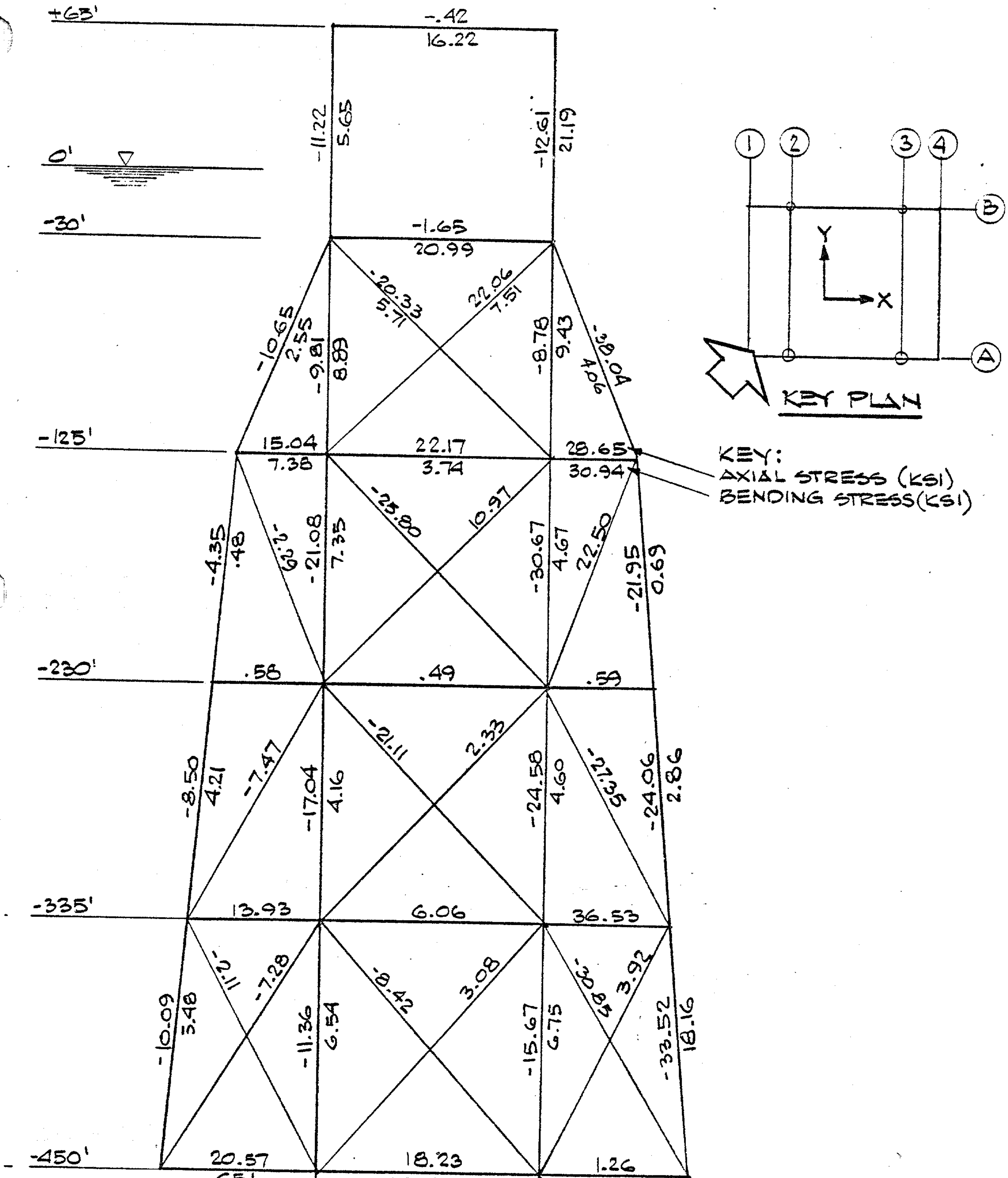
NOMINAL STRESSES ROW B DUE TO LOAD COMBINATION 5

(ST. GEORGE, 300 000 BPD)

FIGURE 4-22



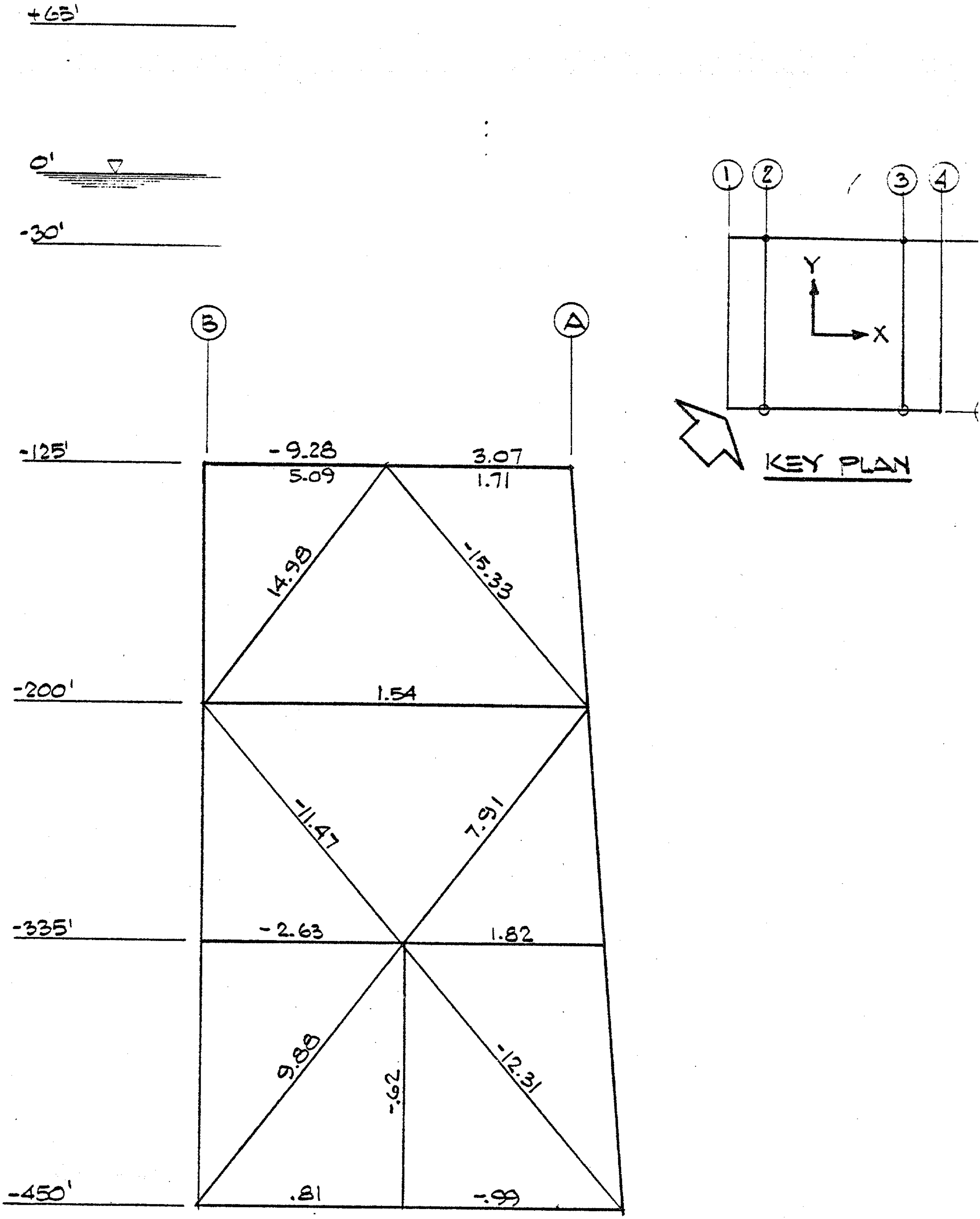
NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 4
DIAGONAL LOADING (NAVARIN 300.000 BPD)



NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION 4

(NAVARIN 300.000 BPD)

FIGURE 4-24

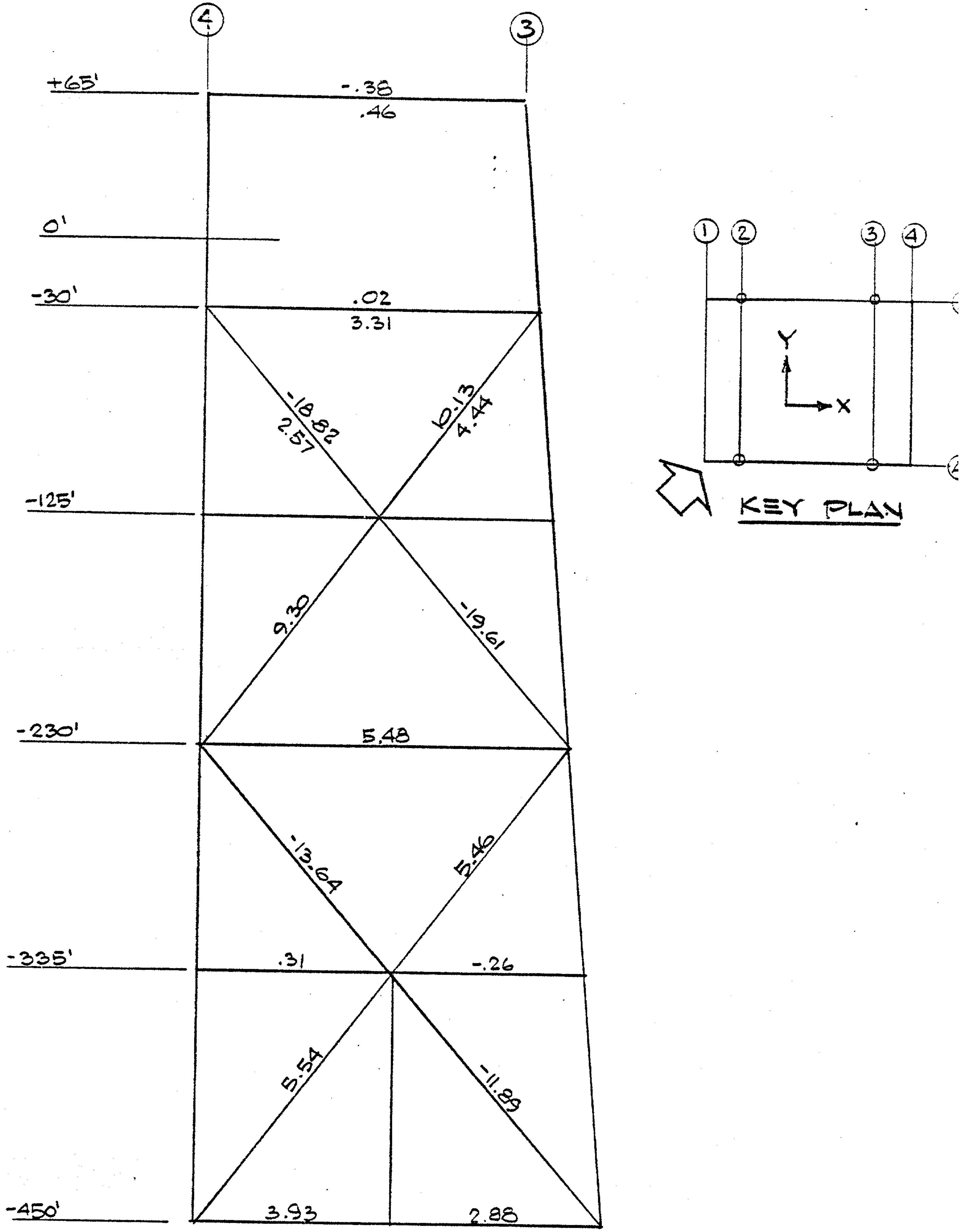


NOMINAL STRESSES ON ROW 1 DUE TO LOAD COMBINATION 4

DIAGONAL LOADING

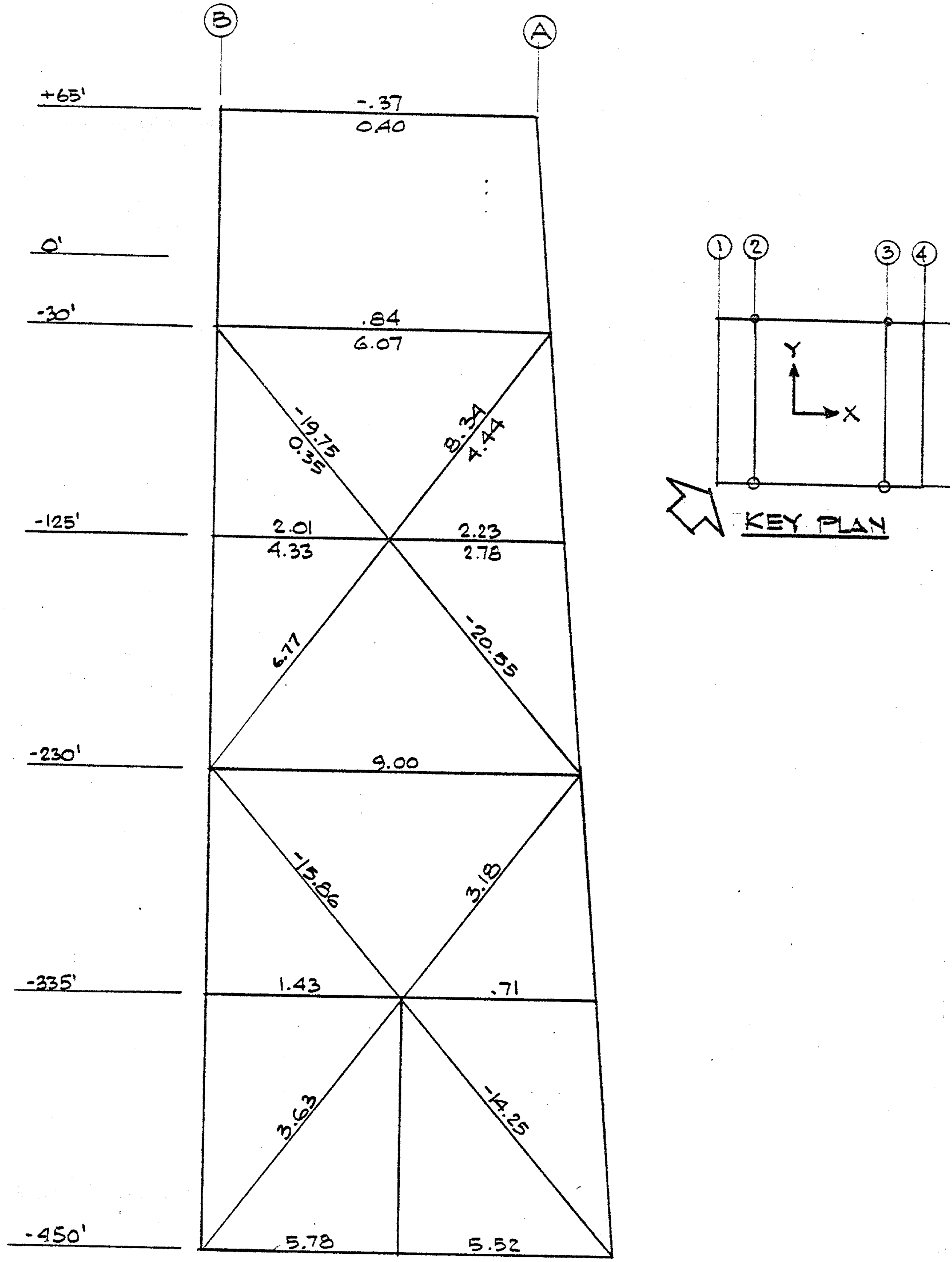
(NAVARIN, 300 000 BPD)

FIGURE 4-25



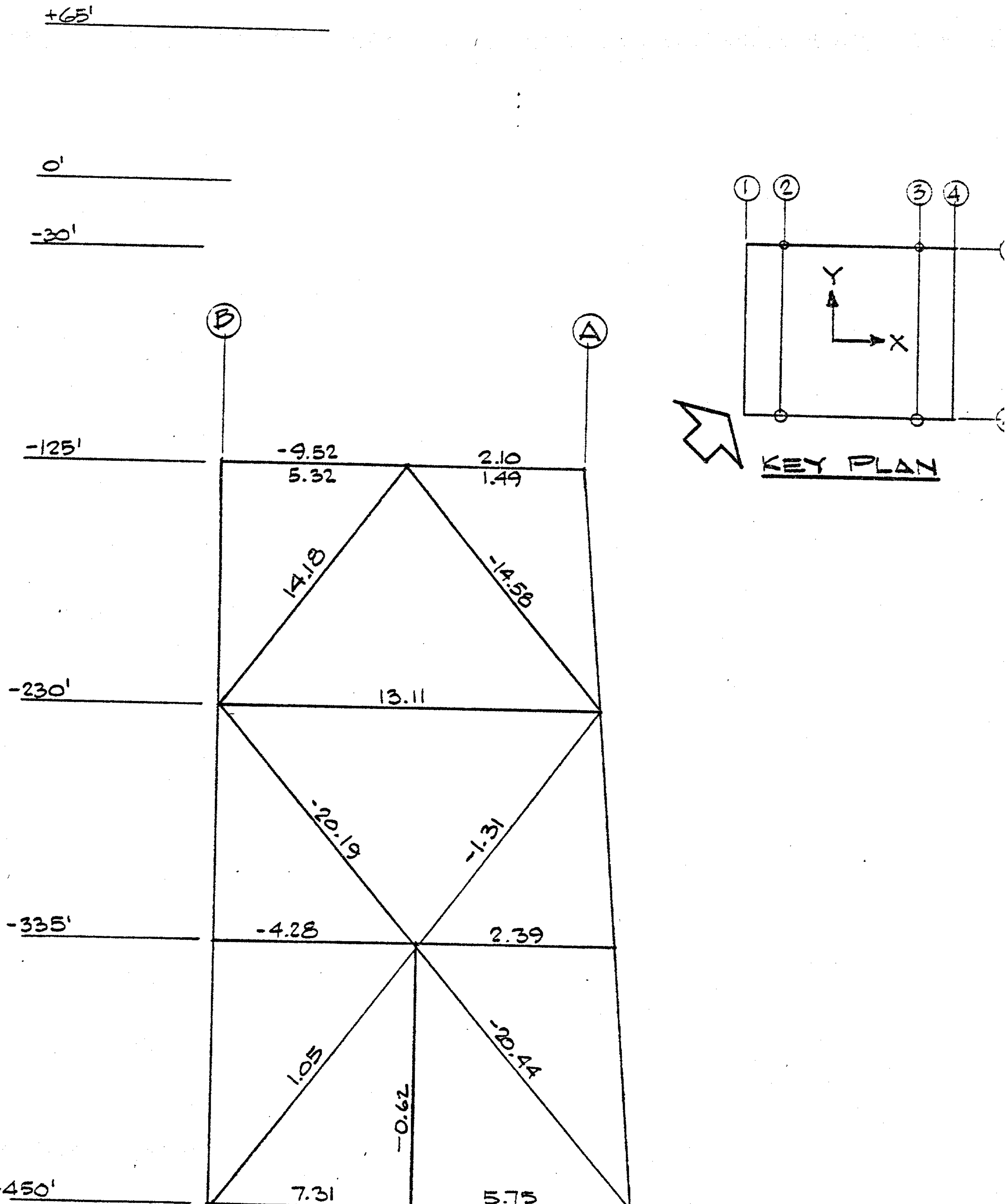
NOMINAL STRESSES ON ROW 2 DUE TO LOAD COMBINATION 4
DIAGONAL LOADING (NAVARIN, 300 000 BPD)

FIGURE 4-26



NOMINAL STRESSES ON ROW 3 DUE TO LOAD COMBINATION 4
DIAGONAL LOADING (NAVARIN, 300.000 BPD)

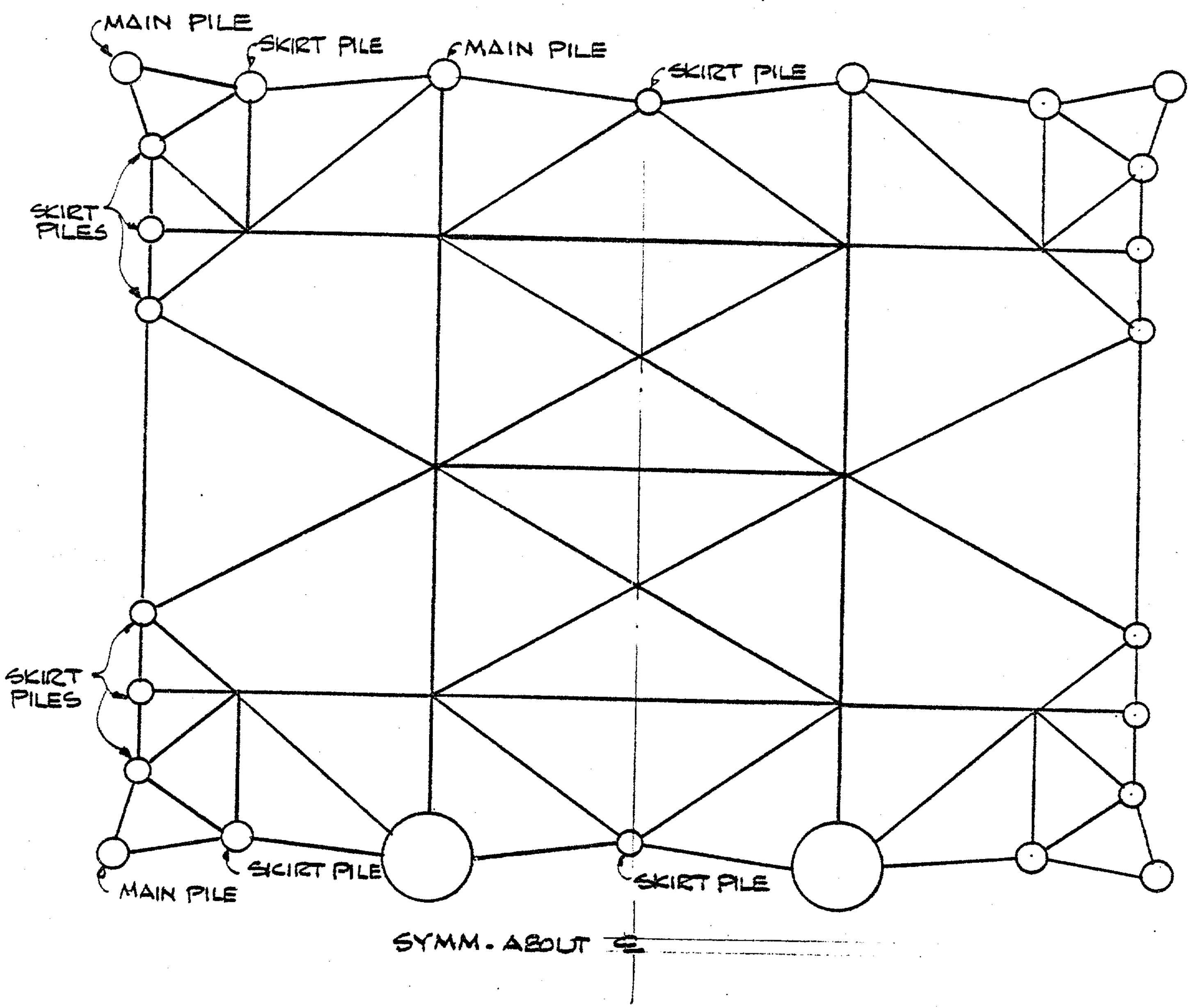
FIGURE 4-27



NOMINAL STRESSES ON ROW 4 DUE TO LOAD COMBINATION 4

DIAGONAL LOADING

(NAVARIN, 300.000 BPD)



MODIFIED FOUNDATION FOR UPPER BOUND WATER AT NAVARIN BASIN

FIGURE 4-29

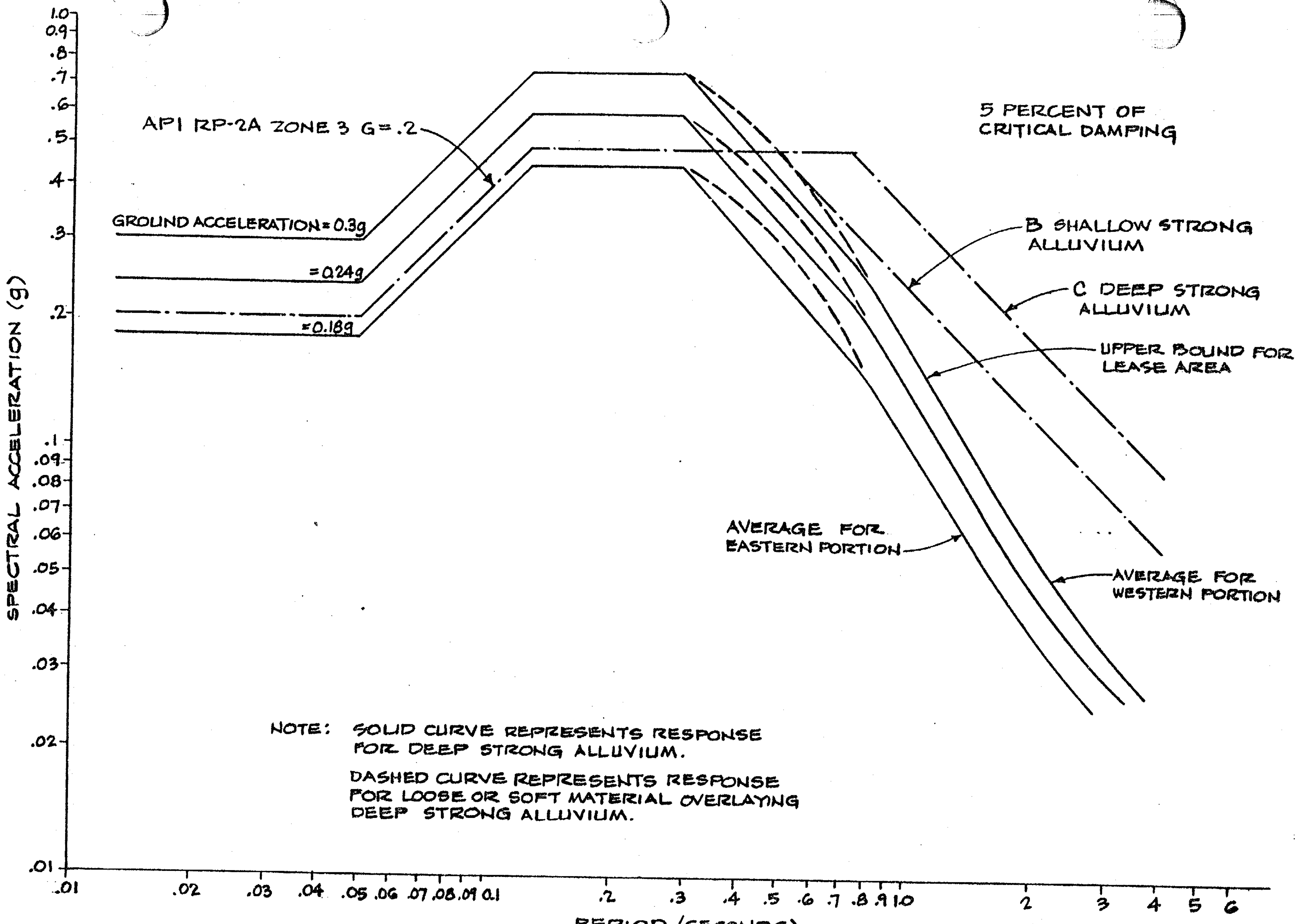
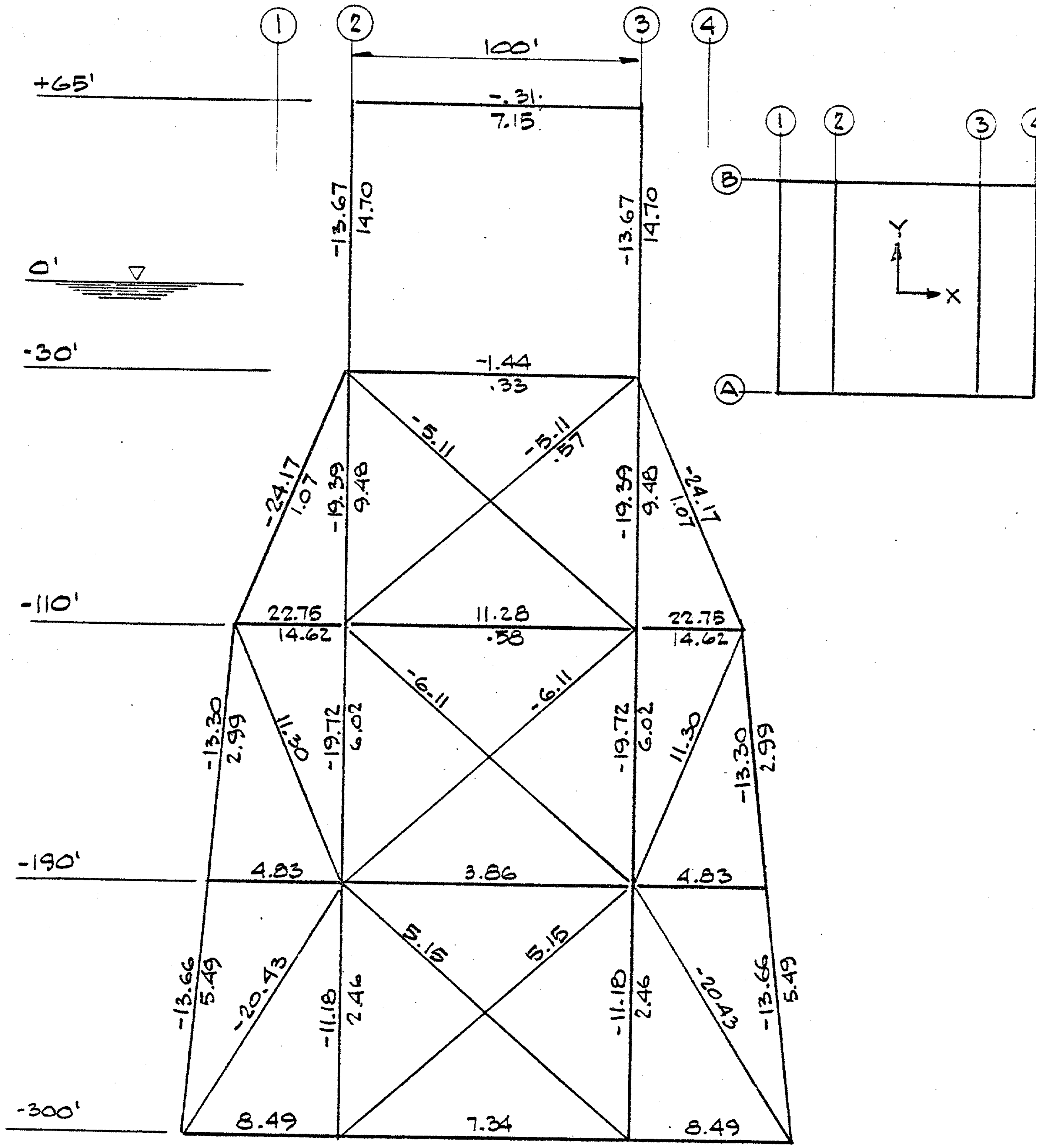
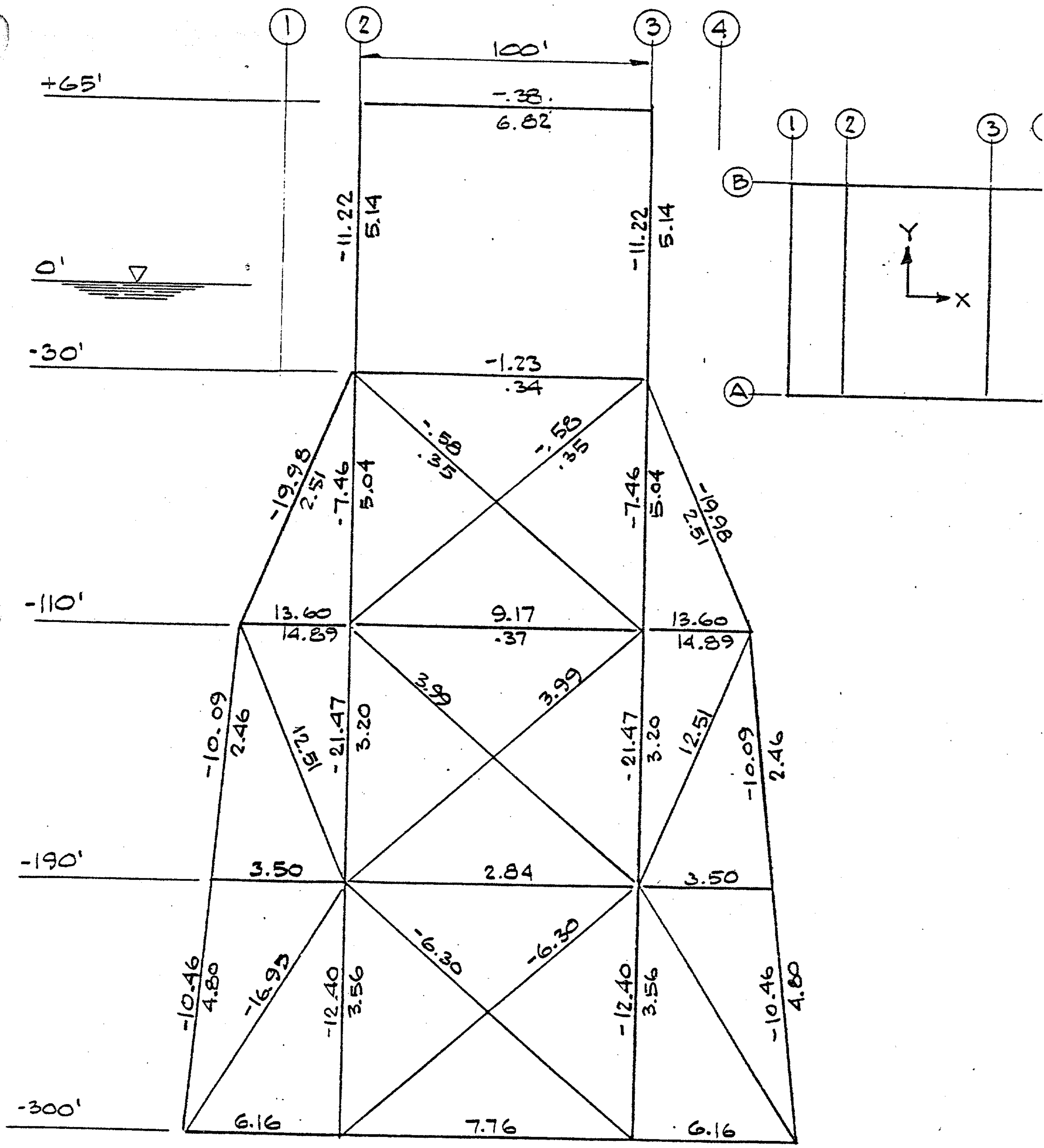


FIGURE 4-30 ACCELERATION RESPONSE SPECTRUM -



NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION I
(NORTH ALEUTIAN SHELF, 300.000 BPD)

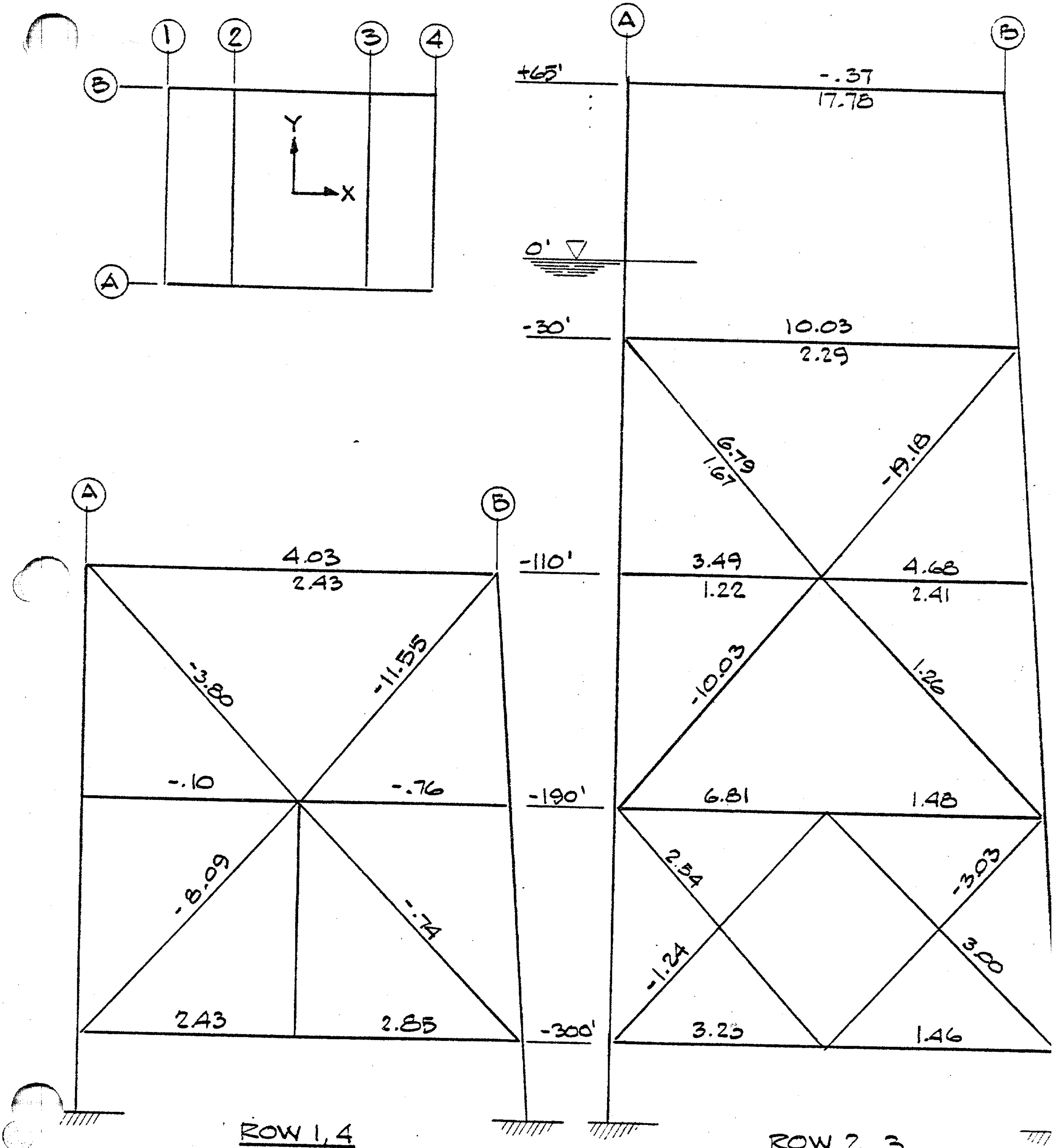
FIGURE 4-31



NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION I

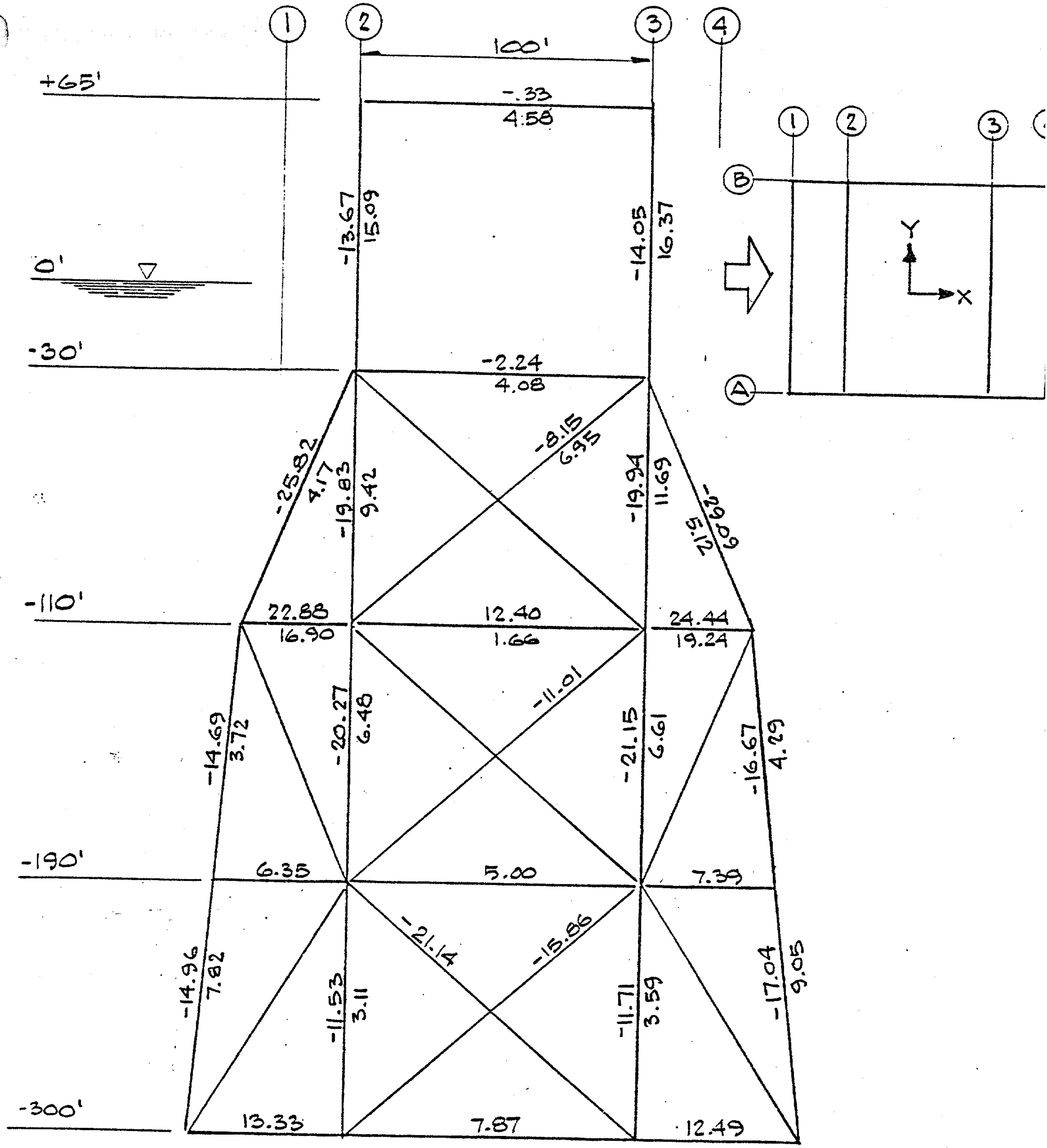
(NORTH ALEUTIAN SHELF 300000 BPD)

FIGURE 4-32

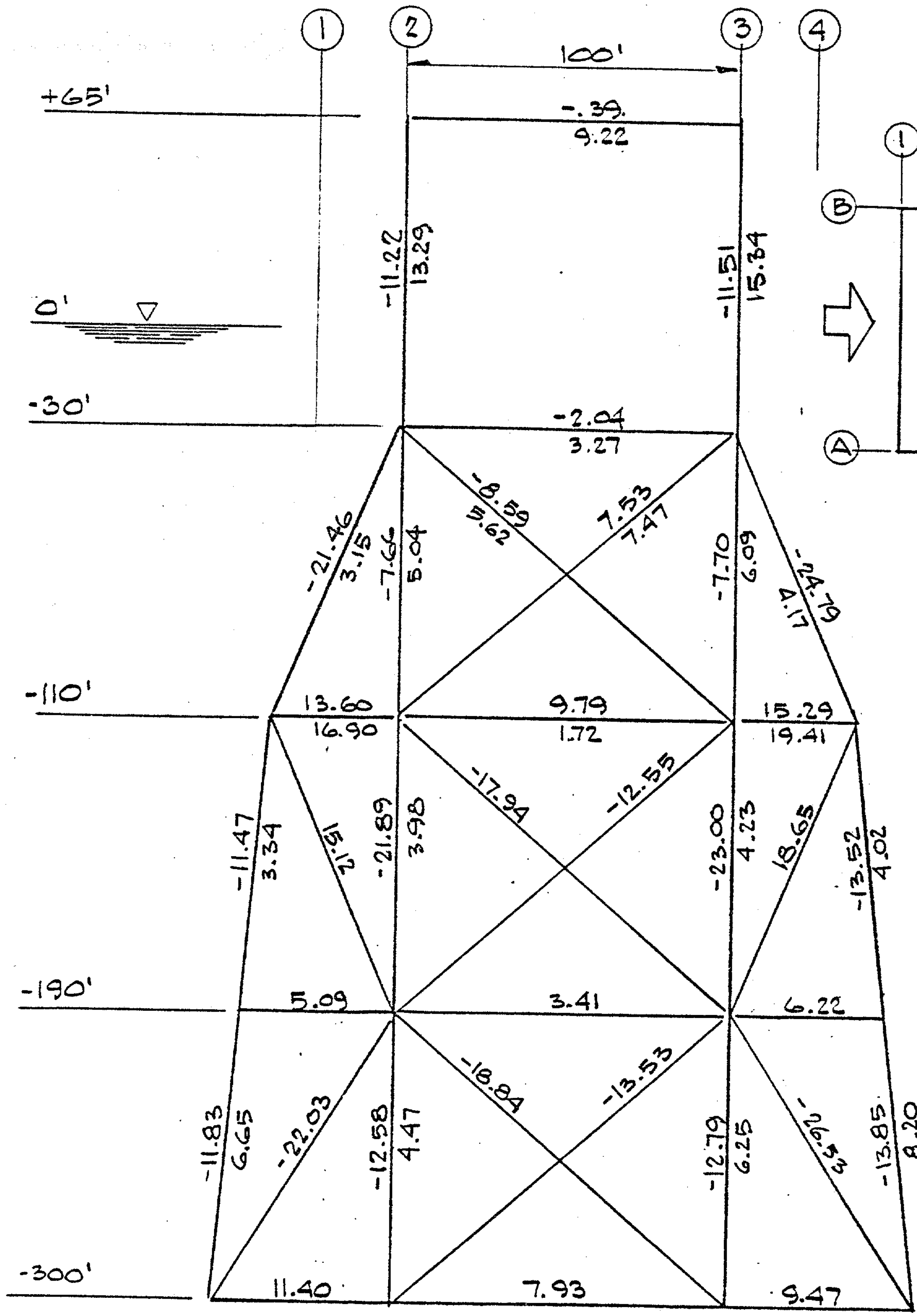


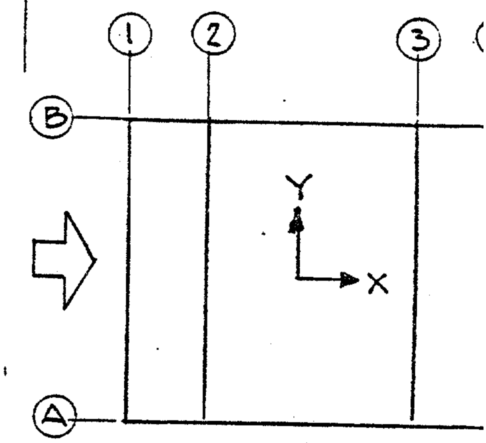
NOMINAL STRESSES ON ROWS 1, 2, 3 & 4 DUE TO LOAD COMBINATION I
(NORTH ALEUTIAN SHELF, 300 000 BPD)

FIGURE 4 - 33



NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 2
WITH STORM IN X-DIRECTION
(NORTH ALEUTIAN SHELF 300000 BPD)
FIGURE 4-34

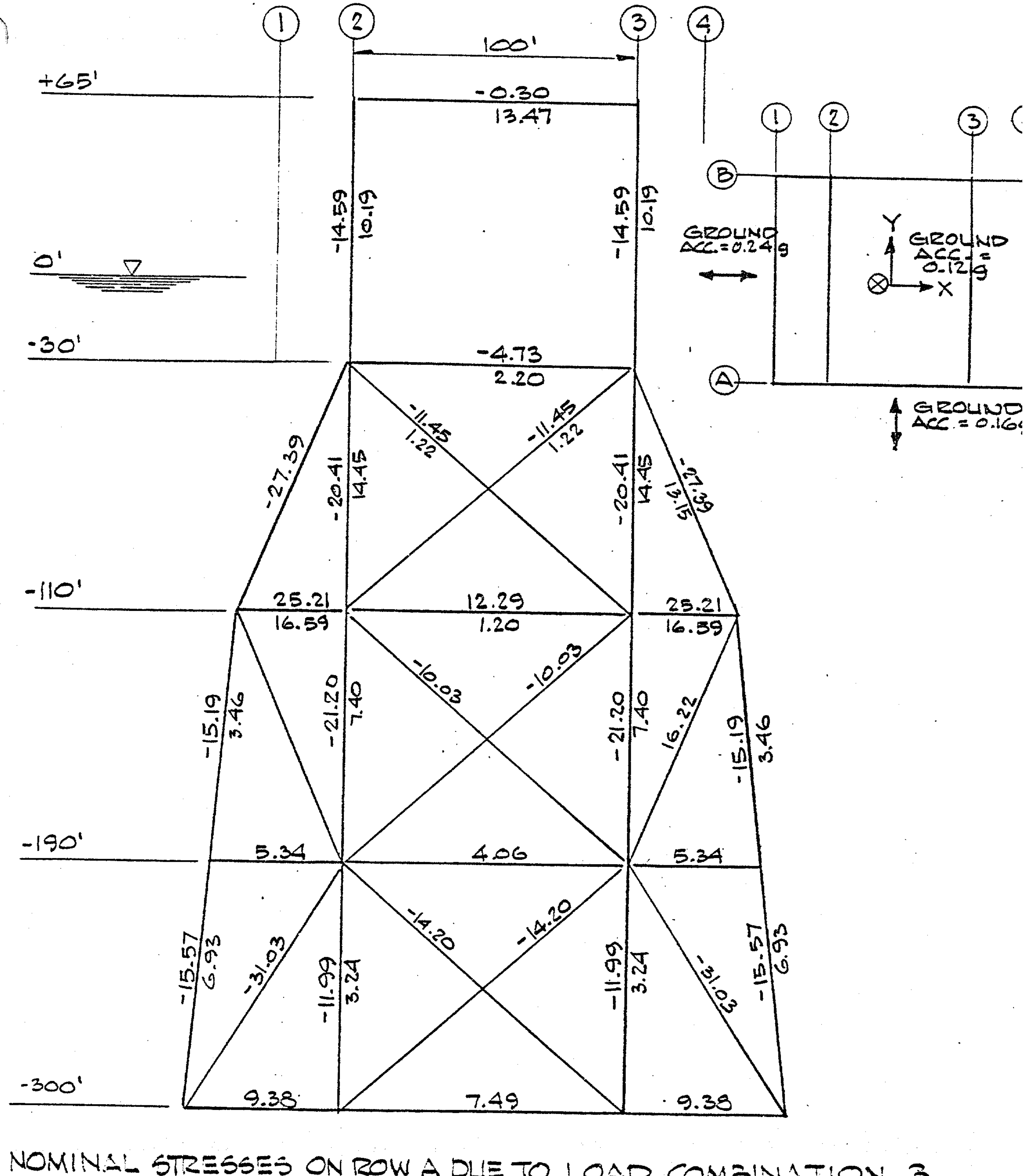




NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION 2 WITH STORM IN X-DIRECTION

(NORTH ALEUTIAN SHELF 300 000 BPD)

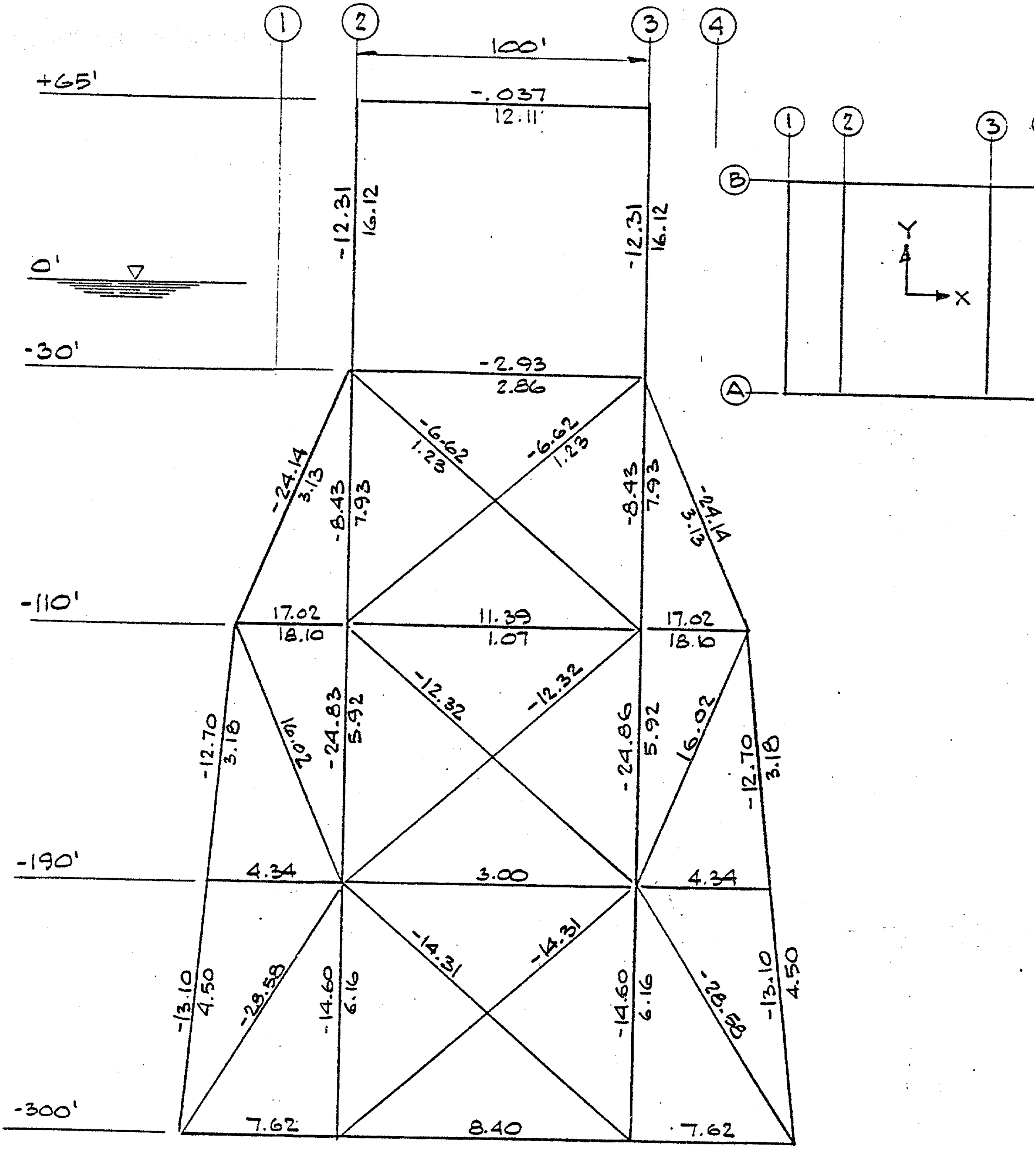
FIGURE 4-35



NOMINAL STRESSES ON ROW A DUE TO LOAD COMBINATION 3

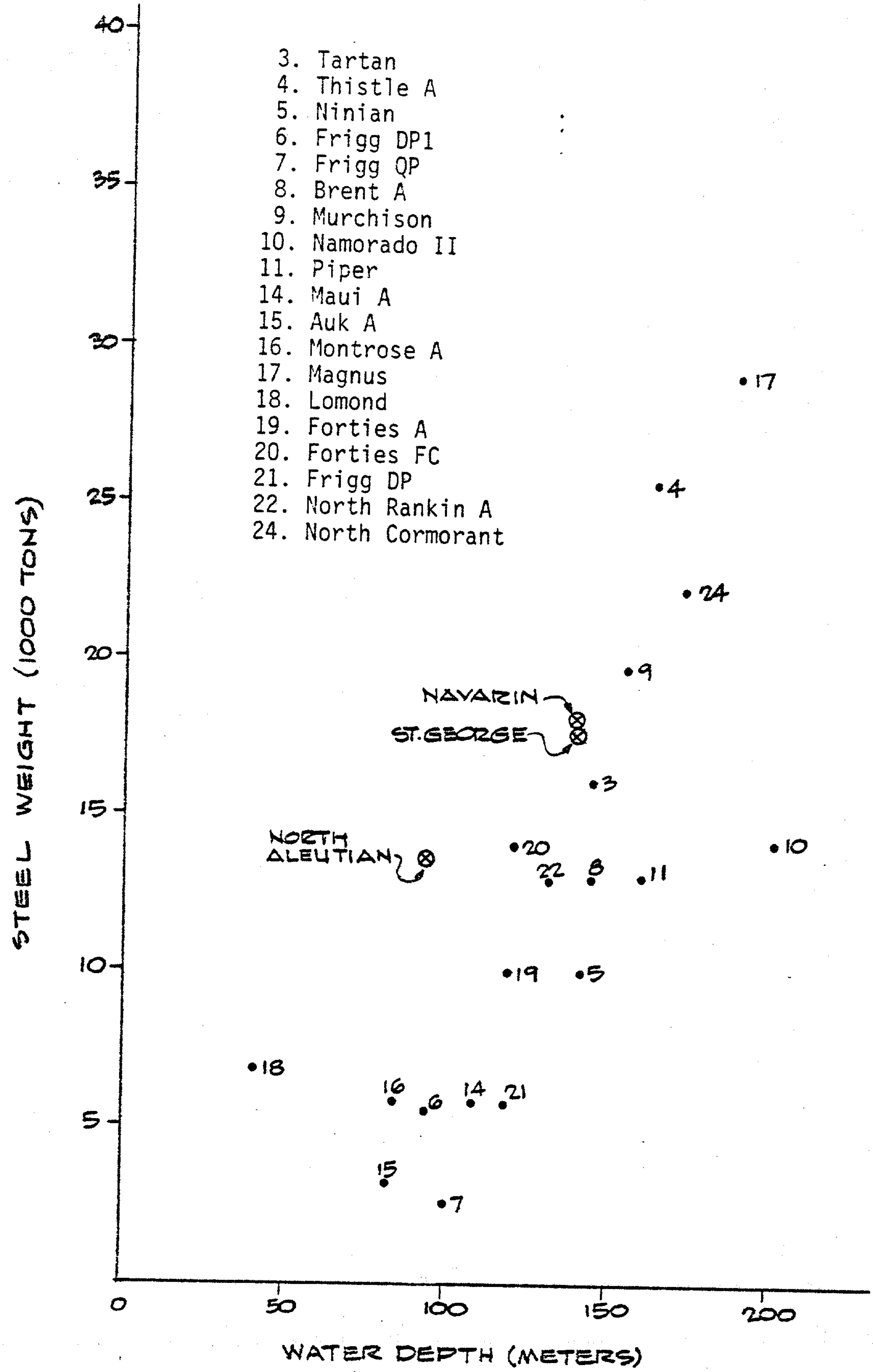
(NORTH ALEUTIAN SHELF 300000 BPD)

FIGURE 4-36



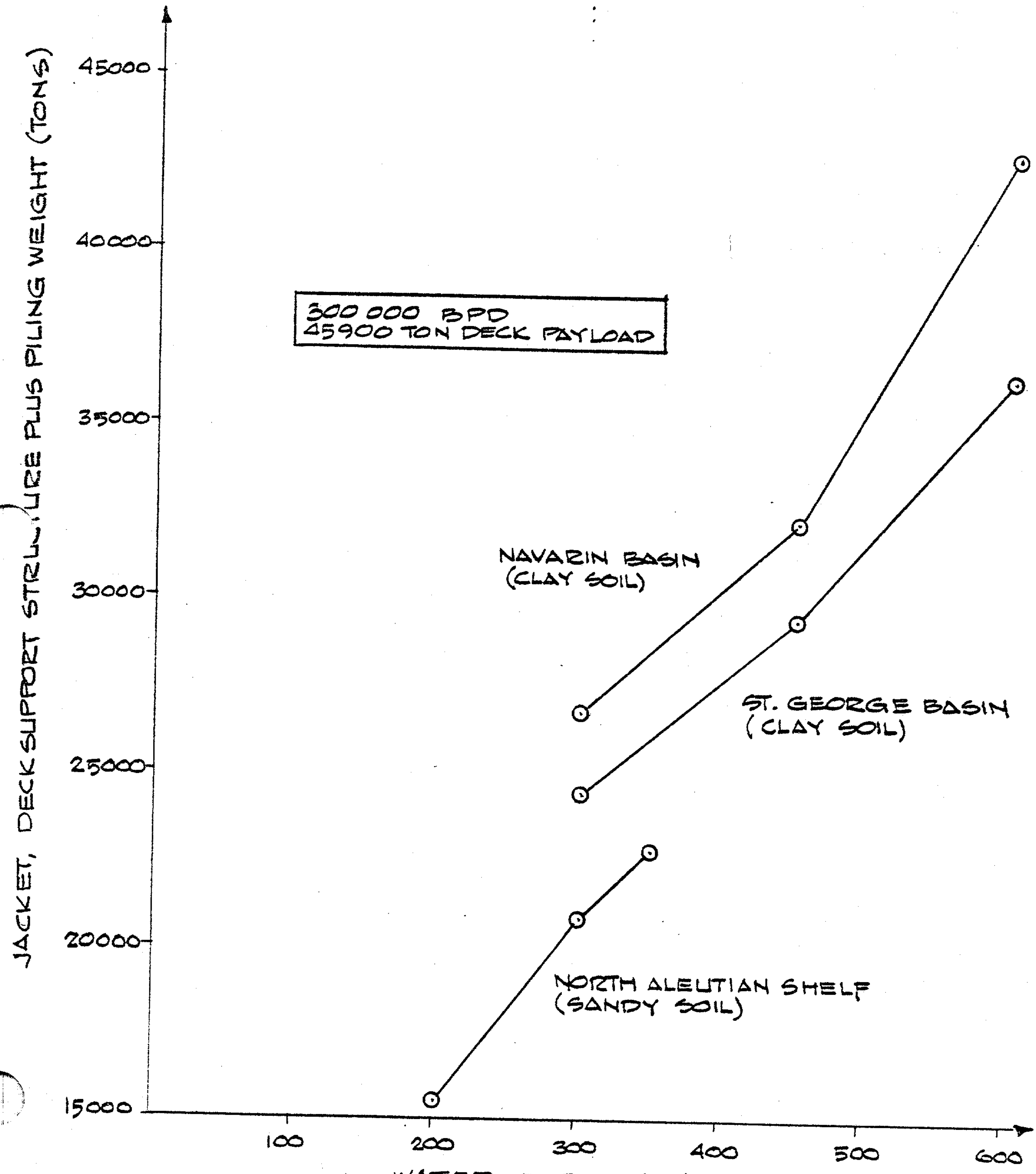
NOMINAL STRESSES ON ROW B DUE TO LOAD COMBINATION 3
(NORTH ALEUTIAN SHELF 300000 BPD)

FIGURE 4-37



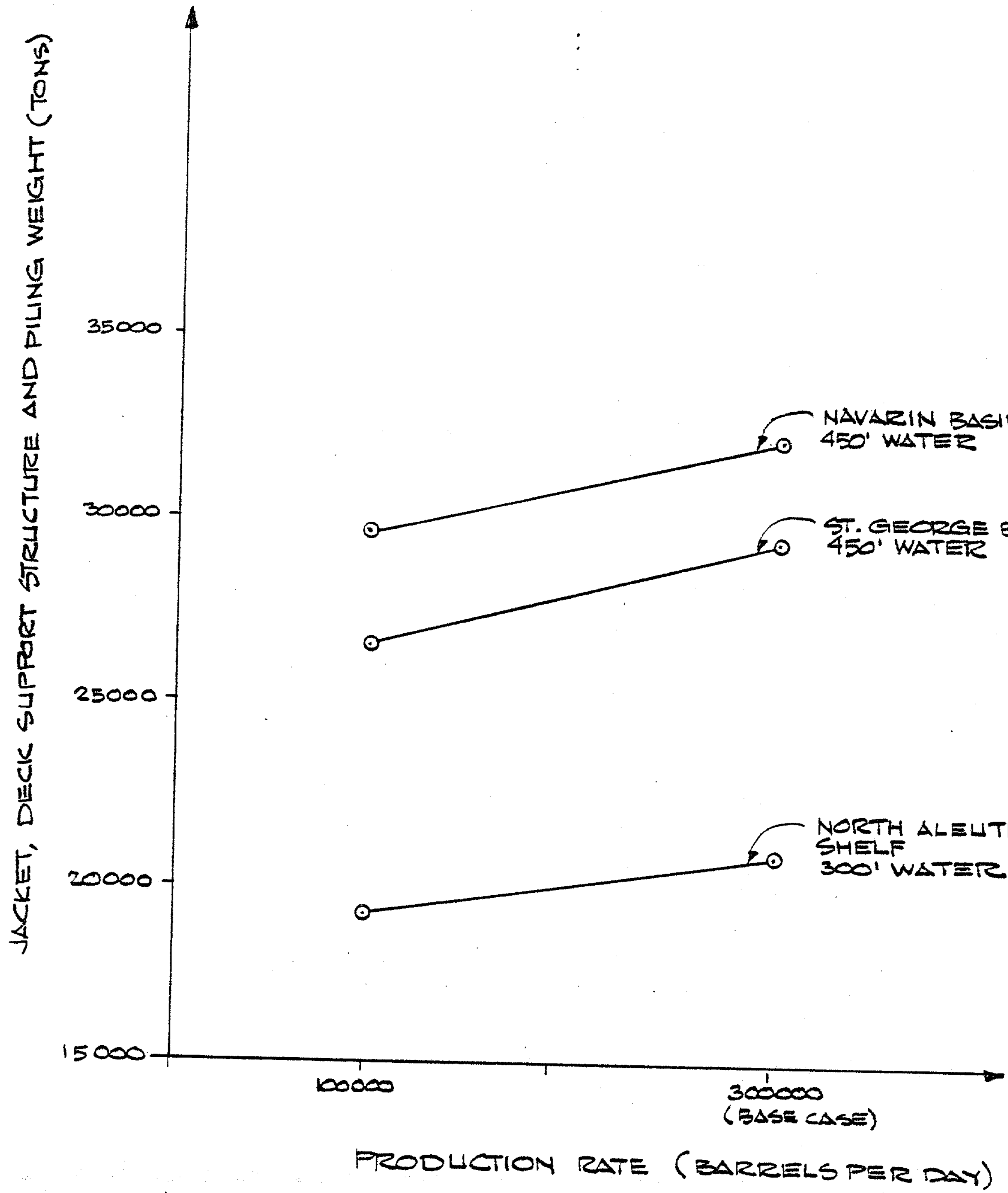
ESTIMATED SOUTHERN BERING SEA STEEL JACKET TONNAGE COMPARED TO ACTUAL NORTH SEA STEEL JACKET TONNAGE

FIGURE 4-38



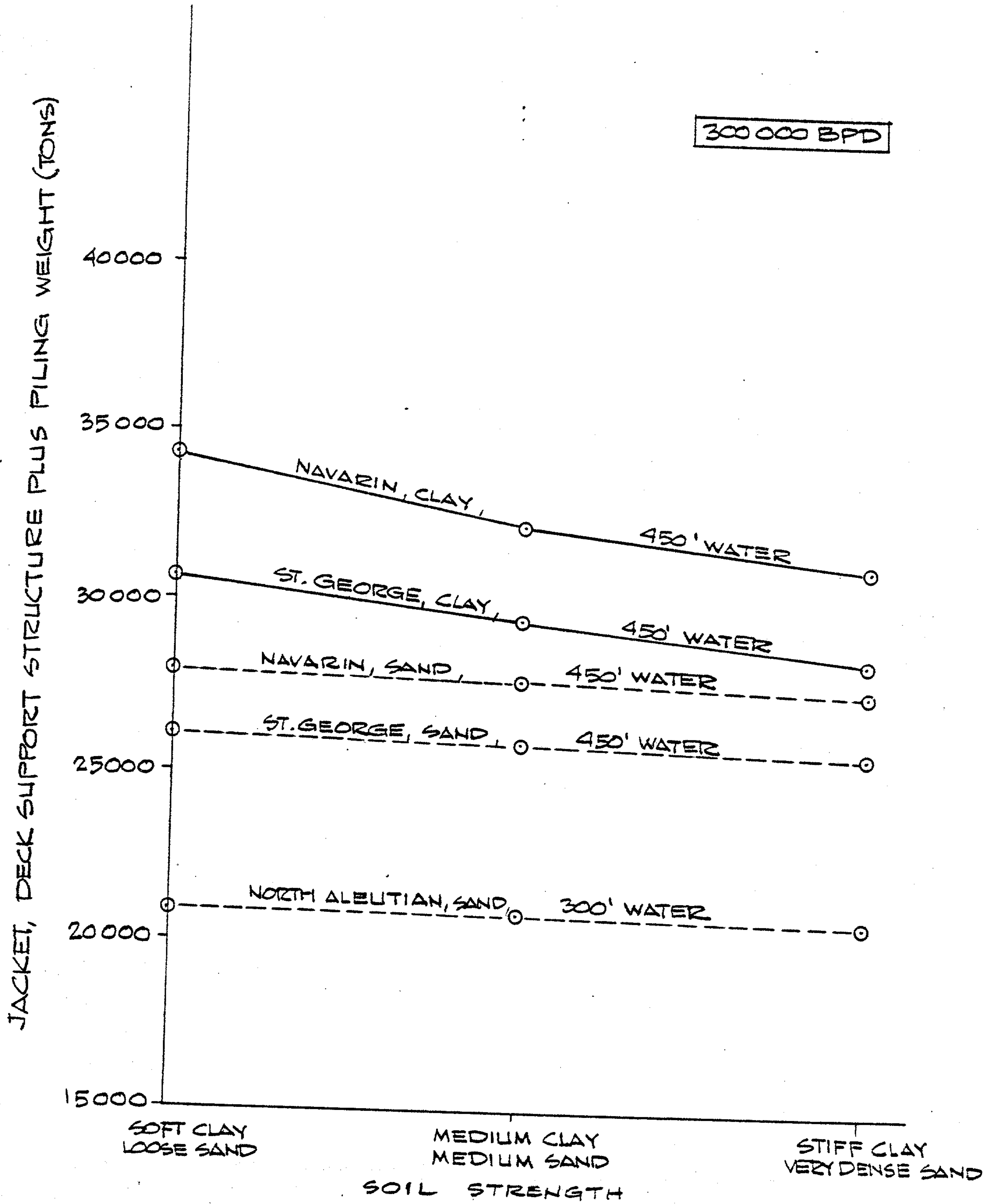
VARIATION OF STRUCTURE WEIGHT WITH WATER DEPTH

FIGURE 4-39



VARIATION OF STRUCTURE WEIGHT WITH PRODUCTION RATE

FIGURE 4-40



VARIATION OF STRUCTURE WEIGHT WITH SOIL STRENGTH

FIGURE 4-41

TABLE 4-1
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: ST. GEORGE BASIN

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Oceanographic			
Water Depth (ft)	450	300	600
Wind Speed (mph)	130	120	140
Wave Height (ft)	85	75	90
Wave Period (sec)	15	14	17
Current Speed - Surface (fps)	4	0	5
Current Speed - Bottom (fps)	0	0	1
Surge and Tide Height (ft)	7	5.5	8.5
Deck Elevation (ft + MWL) (to bottom of support)(1)	64	56.5	68.5
Soil			
Soil Type A (cohesive) Figure 1-5	Medium Clay	Soft Clay	Stiff Clay
Soil Type B (granular) Figure 1-6	Med. Sand.	Loose Sand	Dense Sand

(1)Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 4-1, CONT.

LOCATION: ST. GEORGE BASIN

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Earthquake			
Ground Acceleration (g's)	0.125	0.10	0.15
Ground Velocity (ips)	4.25	3.5	5.0
Ground Displacement (in)	0.5	0.4	1.0
Acceleration Response Spectrum	Figure 1-7	Figure 1-7	Figure 1-7
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	2	2	4
Rafted (Consolidated) Floe Thickness (ft)	6	4	7
Ridging (Including Unconsolidated Ice) Thickness (ft)	15	12	18
Rafted Ice Loading (kips/ft)	85	55	110
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	60	40	90

TABLE 4-2

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES
FOR DESIGN BASIS OF SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
(SEE NOTES ON NEXT PAGE)

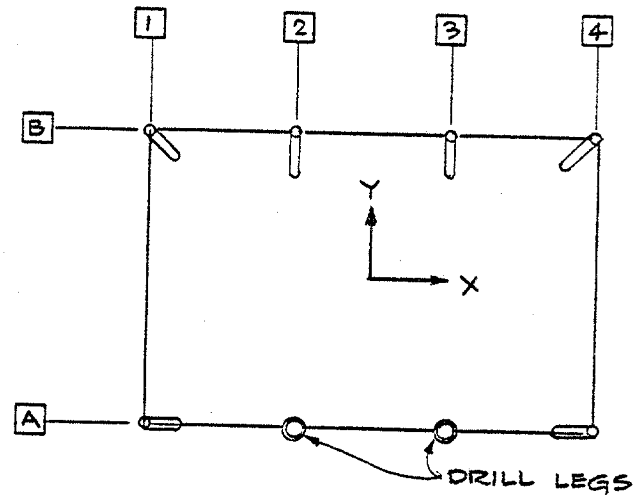
LOCATION: St. George Basin
WATER DEPTH: 450'
PRODUCTION RATE: 300000 B/D

LOAD COMBINATION	PLATFORM SHEAR (KIPS) @ MUDLINE X-AXIS	Y-AXIS	VERTICAL	OVERTURNING MOMENT (KIP-FEET) ABOUT X-AXIS	Y-AXIS	Z-AXIS
1. D+L+B	0	0	107500.	148200.	0	0
2. D+L+B+H+W+C	12550.	11350.	108500.	4020000.	4450000.	132200.
3. D+0.75L+B+E,(Sandy Soil)	3100.	2950.	117200.	810000.	720000.	237000.
*4. D+L+B+I+W	8800.	8800.	107500.	4160000.	4010000.	---
5. D+L+B+S+W	3850.	3850.	108000.	1948000.	1800000.	9800.

REQUIRED PILE GROUP CAPACITIES AT LEG

	A1 and A4	B1 and B4	A2 and A3	B2 and B3
1. D+L+B	42000.	27000.	24000.	15000.
2. D+L+B+H+W+C	47000.	38000.	21000.	17000.
3. D+0.75L+B+E,(Sandy Soil)	34000.	23000.	17000.	11000.
*4. D+L+B+I+W	46000.	37000.	21000.	17000.
5. D+L+B+S+W	37000.	26000.	19000.	12000.

TABLE 4-2, Cont.



1. Units are KIPS or KIP-feet.

2. Listed values are maxima or minima for loading in X, Y or diagonal directions.

3. Reactions for storm and ice loading are assumed virtually independent of soil type.

4. Foundation safety factors were 2.0 for operating conditions and 1.5 for design storm or strength requirement seismic.

5. Load cases are as follows:

   D= Dead load including dry deck equipment
   L= Live loads such as drilling consumables, hook load, etc.
   B= Jacket Buoyancy
   H= Hydrodynamic load due to design wave (water depth includes surge and tide)
   W= Wind Load
   C= Hydrodynamic load due to current (current and wave kinematics combined)
   E= Strength requirement seismic
   I= Rafted ice flow
   S= Local ice pressure on two legs due to ice slam

6. Extrapolated values are indicated by *. Other values are from computer analysis.

7. Piles groups at Legs A1, A4, B1, and B4 each include one main pile and three skirt piles.

TABLE 4-3

NATURAL PERIODS AND MODE SHAPES
450' Water Depth
300,000 Barrel per day case

MODE NO.	PERIOD (sec)	DESCRIPTION OF DOMINANT RESPONSE
1	4.239	First lateral Y-mode
2	3.676	First lateral X-mode
3	2.822	First torsional
4	1.162	Second lateral Y-mode (inflection between El + 65' and El - 30')
5	1.048	Second lateral X-mode (inflection between El + 65' and El - 30')
6	.9061	Second torsional (inflection between El + 65' and El - 30')
7	.8062	Complex + vertical distortion
8	.8025	Complex + third lateral X-mode
9	.6958	Vertical
10	.6425	Third lateral Y-mode (inflections between El + 65' and El - 30', and at El - 230')
11	.5394	Third lateral X-mode (inflections between El + 65' and El - 30', and between El - 125' and El - 230')
12	.5326	Combined X-bending and plan distortion
13	.4938	Combined torsional and X-bending

TABLE 4-4

JACKET WEIGHT FOR ST. GEORGE BASIN
(BASE CASE = 450' WATER, 300000 BPD)

Legs (Below Elevation +20')		
Shell	3600	
Diaphragms and conductor guides	900	
Stiffeners	1200	
Inner Sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		6000
Vertical Bracing		2500
Skirt Pile Sleeves		1700
Horizontal Bracing		1400
Thickening at joint cans, mill tolerance secondary braces and stiffeners @ 20% of above items		2300
Mudmats		1000
Appurtenances		
Launch cradle	900	
J-tubes and risers, sumps	140	
Flooding and grouting systems	100	
Corrosion protection	600	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2500
Total jacket		17400 tons

TABLE 4-5

PILING AND DECK SUPPORT STRUCTURE
WEIGHTS FOR ST. GEORGE BASIN
(BASE CASE = 450' WATER, 300000 BPD)

Piling (90 ø 1.5)	
10300' x 0.71 ton/ft	7310
Thick Wall Sections and Stabbing Guides @ 20%	1460
Skirt Pile Follower	260
	9030 tons
Deck Support Structures	
Box Girders	2330
Horizontal Bracing	20
Legs (above elevation +20)	600
Temporary Supports	30
	2980 tons
Temporary Pile Guide Deck (optional)	
Box Girders	430
Main Leg Pile Guides	140
Other Pile Guides	90
Contingency @ 10%	70
	730 tons

TABLE 4-6

450' W. D. BERING SEA STEEL STRUCTURE

FABRICATOR SUMMARY

	CRANES 150T +	YARD AREA	ROLLING LIMITS	ACCESS FOR 600' BARGE	EXPERIENCE	PERMANENT STAFF	COMMENTS
KAISER - VALLEJO, CA[1]	6	40 AC	15'ODx5"	OK	(1) HONDO "A# 850' W.D. 12,000 T (2) EUREKA 700' W.D. 18,000 T	1100	BRIDGE LIMITS BASE HEIGHT TO 200'
- NAPA, CA[2]		138 AC					
NIPPON STEEL CORPORATION - - WAKAMATSU	6	150 AC	20'ODx6"	OK	IWAKI 580' W.D. 16,000 T	1300	
NIPPON KOKAN (NKK)	10	500 AC	10'ODx3"	OK	(1) N. RANKIN 475' W.D. 16,500 T (2) MAUI A	3000	

[1]Assembly Area

[2]Fabrication of Components

TABLE 4-7

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES
FOR DESIGN BASIS LOADINGS AT VARIOUS WATER DEPTHS
(SEE NOTES ON TABLE 4-2)

LOCATION: St. George Basin
PRODUCTION RATE: 300000 B/D

LOAD COMBINATION	WATER DEPTH	PLATFORM SHEAR (KIPS) @ MUDLINE X-AXIS	Y-AXIS	VERTICAL	OVERTURNING MOMENT (KIP-FEET) ABOUT X-AXIS	Y-AXIS	Z-AXIS
1. D+L+B	*300'	0	0	102000.	140000.	0	0
		0	0	107500.	148200.	0	0
	*600'	0	0	113000.	156000.	0	0
2. D+L+B+H+W+C	*300'	12200.	11000.	103000.	2300000.	2500000.	132000.
	450'	12550.	11350.	108500.	4020000.	4450000.	132200.
	*600'	13000.	11700.	114000.	5900000.	6600000.	132000.

REQUIRED PILE GROUP CAPACITIES AT LEG

		A1 and A4	B1 and B4	A2 and A3	B2 and B3
1. D+L+B	*300'	40000.	26000.	23000.	14000.
	450'	42000.	27000.	24000.	15000.
	*600'	44000.	28000.	25000.	16000.
2. D+L+B+H+W+C	*300'	39000.	30000.	19000.	14000.
	450'	47000.	38000.	21000.	17000.
	*600'	55000.	46000.	23000.	20000.

TABLE 4-8

PLATFORM STRUCTURE WEIGHT FOR VARIOUS
WATER DEPTHS AT ST. GEORGE BASIN (300000 BPD)

	300' W.D.	450' W.D.	600' W.D.
Jacket			
Legs	4500	6000	7900
Vertical Bracing	1700	2500	3300
Skirt Pile Sleaves	1600	1700	1800
Horizontal Bracing	1200	1400	1600
Joint Cans, Secondaries @ 20% (30% for 600' Water)	1800	2300	4300
Mudmats	800	1000	1300
Appurtenances	2100	2500	3000
	13700	17400	23200
Piling (90 ø 1.5)			
Nominal Length and Thickness	6280	7310	8270
Thick Walls and Guides @ 20%	1260	1460	1650
Skirt Pile Follower	150	260	370
	7690	9030	10290 tons
Deck Support Structures	2980	2980	2980
Total	24370 tons	29410 tons	36470 tons

TABLE 4-9

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES FOR DESIGN BASIS OF SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
(SEE NOTES ON TABLE 4-2)

LOCATION: St. George Basin
WATER DEPTH: 450'
PRODUCTION RATE: 100000 B/D

LOAD COMBINATION	PLATFORM SHEAR (KIPS) @ MUDLINE X-AXIS	Y-AXIS	VERTICAL	OVERTURNING MOMENT (KIP-FT) ABOUT X-AXIS	Y-AXIS	Z-AXIS
1. D+L+B	0	0	78840.	148200.	0	0
*2. D+L+B+H+W+C	10000.	9200.	79800.	3300000.	3600000.	110000.
*3. D+0.75L+B+E,(Sandy Soil)	3100.	2900.	89000.	810000.	720000.	237000.
4. D+L+B+I+W	5500.	5500.	78840.	2500000.	2500000.	-
*5. D+L+B+S+W	3850.	3850.	79000.	1948000.	1800000.	9800.

REQUIRED PILE GROUP CAPACITIES AT LEG

LOAD COMBINATION	A1 and A4	B1 and B4	A2 and A3	B2 and B3
1. D+L+B	31000.	20000.	18000.	11000.
*2. D+L+B+H+W+C	36000.	30000.	16000.	13000.
*3. D+0.75L+B+E,(Sandy Soil)	25000.	18000.	13000.	8000.
*4. D+L+B+I+W	32000.	25000.	15000.	12000.
*5. D+L+B+S+W	29000.	21000.	14000.	8900.

TABLE 4-10

JACKET WEIGHT FOR 100000 BPD PRODUCTION
AT ST. GEORGE BASIN (450' WATER)

Legs		
Shell	3200	
Diaphragms and conductor guides	650	
Stiffeners	750	
Inner Sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		4900
Vertical Bracing		2100
Skirt Pile Sleeves		1700
Horizontal Bracing		1400
Thickening at joint cans, mill tolerance, secondary braces and stiffeners @ 20%		2000
Mudmats		900
Appurtenances		
Launch cradle	900	
J-tubes and risers, sumps	140	
Flooding and grouting systems	100	
Corrosion protection	600	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2500
Total jacket		15500 tons

TABLE 4-11

PILING AND DECK SUPPORT STRUCTURE
WEIGHTS FOR 100000 BPD PRODUCTION
AT ST. GEORGE BASIN (450' WATER)

Piling (90 ø 1.5)	
9740' x 0.71 ton/ft	6920
Thick Wall Sections and Stabbing Guides @ 20%	1380
Skirt Pile Follower	260
	8560 tons
Deck Support Structures	
Box Girders	2000
Horizontal Bracing	20
Legs (above elevation +20)	380
Temporary Supports	30
	2430 tons
Temporary Pile Guide Deck (optional)	
Box Girders	430
Main Leg Pile Guides	140
Other Pile Guides	90
Contingency @ 10%	70
	730 tons

TABLE 4-12

PLATFORM STRUCTURE WEIGHT FOR VARIOUS SOILS
AT ST. GEORGE BASIN (300000 BPD, 450' WATER)

	Clay			Sand		
	Lower	Design	Upper	Lower	Design	Upper
Jacket*	17400	17400	17400	17400	17400	17400
Piling						
Nominal length and thickness	8380	7310	6390	4470	4330	4190
Thick walls and guides @ 20%	1680	1460	1280	890	870	840
Skirt pile follower	260	260	260	260	260	260
	10320	9030	7930	5620	5460	5290
Deck Support Structure*	2980	2980	2980	2980	2980	2980
Total (tons)	30700	29410	28310	26000	25840	25670

*These items are the same as base case.
See Tables 4-4 and 4-5 for details.

TABLE 4-13
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: NAVARIN BASIN

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Oceanographic			
Water Depth (ft)	450	300	600
Wind Speed (mph)	130	120	140
Wave Height (ft)	80	70	85
Wave Period (sec)	14	13	15
Current Speed - Surface (fps)	4	0	5
Current Speed - Bottom (fps)	0	0	1
Surge and Tide Height (ft)	5	4	7
Deck Elevation (ft + MWL)(1)	59	52	64
Soil			
Soil Type A (cohesive) Figure 1-5	Med. Clay	Soft Clay	Stiff Clay
Soil Type B (granular) Figure 1-6	Med. Sand	Loose Sand	Dense Sand

(1)Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 4-13, CONT.

LOCATION: NAVARIN BASIN

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Earthquake			
Ground Acceleration (g's)	0.05	0.01	0.10
Ground Velocity (ips)	1.5	1.0	2.0
Ground Displacement (in)	---	---	---
Acceleration Response Spectrum	Figure 1-8	Figure 1-8	Figure 1-8
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	4	2	5
Rafted (Consolidated) Floe Thickness (ft)	8	6	9
Ridging (Including Unconsolidated Ice) Thickness (ft)	20	15	25
Rafted Ice Loading (kips/ft)	115	80	140
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	110	60	170

TABLE 4-[illegible]

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES
FOR DESIGN BASIS OF SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
(SEE NOTES ON TABLE 4-2)

LOCATION: Navarin Basin
WATER DEPTH: 450'
PRODUCTION RATE: 300000 B/D

	LOAD COMBINATION	PLATFORM SHEAR (KIPS) @ MUDLINE			OVERTURNING MOMENT (KIP-FEET) ABOUT		
		X-AXIS	Y-AXIS	VERTICAL	X-AXIS	Y-AXIS	Z-AXIS
1.	D+L+B	0	0	107500.	148200.	0	0
*2.	D+L+B+H+W+C	12000.	11000.	108000.	3800000.	4200000.	120000.
*3.	D+0.75L+B+E,(Sandy Soil)	1200.	1200.	110000.	320000.	290000.	95000.
4.	D+L+B+I+W	14290.	14310.	108000.	6280000.	6480000.	42300.
5.	D+L+B+S+W	3850.	3850.	108000.	1948000.	1800000.	9800.

REQUIRED PILE GROUP CAPACITIES AT LEG

		A1 and A4	B1 and B4	A2 and A3	B2 and B3
1.	D+L+B	42000.	27000.	24000.	15000.
*2.	D+L+B+H+W+C	46000.	37000.	21000.	16000.
*3.	D+0.75L+B+E,(Sandy Soil)	32000.	21000.	16000.	10000.
4.	D+L+B+I+W	69000.	59000.	30000.	22000.
5.	D+L+B+S+W	37000.	26000.	19000.	12000.

TABLE 4-15

JACKET WEIGHT FOR NAVARIN BASIN
(BASE CASE = 450' WATER, 300000 BPD)

Legs		
Shell	3800	
Diaphragms and conductor guides	900	
Stiffeners	1200	
Inner sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		6200
Vertical Bracing		2800
Skirt Pile Sleeves		1700
Horizontal Bracing		1400
Thickening at joint cans, mill tolerance secondary braces and stiffeners @ 20%		2400
Mudmats		1000
Appurtenances		
Launch cradle	900	
J-tubes and risers, sumps	140	
Flooding and grouting systems	100	
Corrosion protection	600	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2500
Total jacket		18000 tons

TABLE 4-16

PILING AND DECK SUPPORT STRUCTURE WEIGHTS
FOR NAVARIN BASIN
(BASE CASE = 450' WATER, 300000 BPD)

Piling (90 ø 1.75)	
11150' x 0.82 ton/ft	9140
Thick Wall Sections and Stabbing Guides @ 20%	1830
Skirt Pile Follower	300
	11270 tons
Deck Support Structures	
Box Girders	2330
Horizontal Bracing	20
Legs (above elevation +20)	600
Temporary Supports	30
	2980 tons
Temporary Pile Guide Deck (optional)	
Box Girders	430
Main Leg Pile Guides	140
Other Pile Guides	90
Contingency @ 10%	70
	730 tons

TABLE 4-17

PLATFORM STRUCTURE WEIGHT FOR VARIOUS
WATER DEPTHS AT NAVARIN BASIN (300000 BPD)

	300' W.D.	450' W.D.	600' W.D.
Jacket			
Legs	4600	6200	8200
Vertical Bracing	1900	2800	3800
Skirt Pile Sleaves	1600	1700	2600
Horizontal Bracing	1200	1400	1700
Joint Cans, Secondaries @ 20% (26% for 600' Water)	1900	2400	4300
Mudmats	800	1000	1400
Appurtenances	1900	2500	3000
	13900	18000	25000
Piling (90 ø 1.75)			
Nominal Length and Thickness	8120	9140	12040
Thick Walls and Guides @ 20%	1620	1830	2410
Skirt Pile Follower	170	300	430
	9910	11270	14880 tons
Deck Support Structures	2980	2980	2980
Total	26790 tons	32250 tons	42860 tons

TABLE 4-18

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES FOR DESIGN BASIS OF SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
(SEE NOTES ON TABLE 4-2)

LOCATION: Navarin Basin
WATER DEPTH: 450'
PRODUCTION RATE: 100000 B/D

	LOAD COMBINATION	PLATFORM SHEAR (KIPS) @ MUDLINE			OVERTURNING MOMENT (KIP-FEET) ABOUT		
		X-AXIS	Y-AXIS	VERTICAL	X-AXIS	Y-AXIS	Z-AXIS
1.	D+L+B	0	0	78840.	148200.	0	0
*2.	D+L+B+H+W+C	9600.	8700.	79000.	3100000.	3400000.	101000.
*3.	D+0.75L+B+E,(Soil A)	1200.	1200.	83000.	320000.	290000.	95000.
*4.	D+L+B+I+W	14290.	14310.	79000.	6280000.	6480000.	42300.
*5.	D+L+B+S+W	3850.	3850.	79000.	1948000.	1800000.	9800.

REQUIRED PILE GROUP CAPACITIES AT LEG

		A1 and A4	B1 and B4	A2 and A3	B2 and B3
1.	D+L+B	31000.	20000.	18000.	11000.
*2.	D+L+B+H+W+C	35000.	29000.	16000.	12000.
*3.	D+0.75L+B+E,(Soil A)	23000.	16000.	12000.	7400.
*4.	D+L+B+I+W	61000.	53000.	25000.	19000.
*5.	D+L+B+S+W	29000.	21000.	14000.	8900.

FIGURE 4-19

JACKET WEIGHT FOR 100000 BPD PRODUCTION
AT ST. GEORGE BASIN (450' WATER)

Legs		
Shell	3300	
Diaphragms and conductor guides	650	
Stiffeners	750	
Inner Sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		5000
Vertical Bracing		2500
Skirt Pile Sleeves		1700
Horizontal Bracing		1400
Thickening at joint cans, mill tolerance, secondary braces and stiffeners @ 20%		2100
Mudmats		900
Appurtenances		
Launch cradle	900	
J-tubes and risers, sumps	140	
Flooding and grouting systems	100	
Corrosion protection	600	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2500
Total jacket		16100 tons

TABLE 4-20

PILING AND DECK SUPPORT STRUCTURE WEIGHTS
FOR 100000 BPD IN NAVARIN BASIN
(450' WATER)

Piling (90 ø 1.75)

10940' x 0.82 ton/ft	8970
Thick Wall Sections and Stabbing Guides @ 20%	1790
Skirt Pile Follower	300
	11060 tons

Deck Support Structures

Box Girders	2000
Horizontal Bracing	20
Legs (above elevation +20)	380
Temporary Supports	30
	2430 tons

TABLE 4-21

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES FOR LOAD COMBINATION 4 WITH LOWER, DESIGN AND UPPER BOUND RAFTED ICE LOADING
(NAVARIN BASIN, 300000 BPD, 450' WATER)
(SEE NOTES ON TABLE 4-2)

	LOWER	DESIGN	UPPER
X-shear (kips)	9900.	14290.	17400.
Y-shear (kips)	10000.	14310.	17400.
Vertical force (kips)	107800.	108000.	108100.
Moment about X-axis (kip-feet)	4370000.	6280000.	7650000.
Moment about Y-axis (kip-feet)	4510000.	6480000.	7890000.
Torsion (kip-feet)	29000.	42300.	51000.
Required Pile Capacity A1 & A4 (kips)	58000.	69000.	77000.
Required Pile Capacity B1 & A4 (kips)	47000.	59000.	67000.
Required Pile Capacity A2 & A3 (kips)	26000.	30000.	33000.
Required Pile Capacity B2 & A3 (kips)	19000.	22000.	24000.

TABLE 4-22

PLATFORM STRUCTURE WEIGHT FOR VARIATION
IN ICE LOADS AT NAVARIN BASIN
(450' WATER, 300000 BPD)

	300' W.D.	450' W.D.	600' W.D.
Jacket			
Legs	6000	6200	6300
Vertical Bracing	2500	2800	3200
Skirt Pile Sleaves	1700	1700	2700
Horizontal Bracing	1400	1400	2600
Joint Cans, Secondaries @ 20%	2300	2400	3000
Mudmats	1000	1000	1200
Appurtenances	2500	2500	2500
	17400	18000	21500
Piling (90 ø 1.75)			
Nominal Length and Thickness	8800	9140	11000
Thick Walls and Guides @ 20%	1760	1830	2200
Skirt Pile Follower	300	300	300
	10860	11270	13500 tons
Deck Support Structures	2980	2980	2980
Total	31240 tons	32250 tons	37980 tons

TABLE 4-23

PLATFORM STRUCTURE WEIGHT FOR VARIOUS SOILS
AT NAVARIN BASIN (300000 BPD, 450' WATER)

	Clay			Sand		
	Lower	Design	Upper	Lower	Design	Upper
Jacket*	18000	18000	18000	18000	18000	18000
Piling (90 ø 1.75)						
Nominal length and thickness	10900	9140	8200	5500	5330	5170
Thick walls and guides @ 20%	2200	1830	1640	1100	1070	1030
Skirt pile follower	300	300	300	300	300	300
	13400	11270	10140	6900	6700	6500
Deck Support Structure*	2980	2980	2980	2980	2980	2980
Total (tons)	34380	32250	31120	27880	27680	27480

*These items are the same as base case.
See Tables 4-15 and 4-16 for details.

TABLE 4-24
DESIGN BASIS AND SENSITIVITY RANGE PARAMETERS FOR
STUDY OF SOUTHERN BERING SEA PRODUCTION SYSTEMS

LOCATION: NORTH ALEUTIAN SHELF

Parameter	Design Basis	Sensitivity Range	
		Lower	Upper
Oceanographic			
Water Depth (ft)	300	200	350
Wind Speed (mph)	120	110	130
Wave Height (ft)	78	65	85
Wave Period (sec)	15	14	16
Current Speed - Surface (fps)	5	0	6
Current Speed - Bottom (fps)	2	0	3
Surge and Tide Height (ft)	12	8	15
Deck Elevation (ft + MWL)(1)	65	53	72
Soil			
Soil Type A (cohesive) Figure 1-5	N/A	N/A	N/A
Soil Type B (granular) Figure 1-6	Med. Sand	Loose Sand	Dense Sand

(1) Deck elevation = 0.6(Wave Height) + Surge and Tide Height + 6 ft. Air Gap.

TABLE 4-[illegible], CONT.

LOCATION: NORTH ALEUTIAN SHELF

Parameter	Design Basis	Sensitivity Range Lower	Sensitivity Range Upper
Earthquake			
Ground Acceleration (g's)	0.24	0.18	0.3
Ground Velocity (ips)	8	6.5	10
Ground Displacement (in)	1.5	1.0	2
Acceleration Response Spectrum	Figure 1-9	Figure 1-9	Figure 1-9
Ice			
Superstructure Icing (in)	4	2	6
Ice Slam Velocity (fps)	12	10	20
Local Ice Pressure (psi over 5 ft sq area)	450	350	600
Ice Crushing Strength - Granular (psi)	250	240	300
Ice Crushing Strength - Columnar (psi)	160	150	170
Ice Floe Thickness (ft)	3	1	4
Rafted (Consolidated) Floe Thickness (ft)	6	4	7
Ridging (Including Unconsolidated Ice) Thickness (ft)	15	12	18
Rafted Ice Loading (kips/ft)	85	55	110
Ridge (Double Shear) Ice Loading (30 ft. diam.) (kips/ft)	60	40	90

TABLE 4-25

BASE (MUDLINE) SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE GROUP CAPACITIES FOR DESIGN BASIS OF SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
(SEE NOTES ON TABLE 4-2)

LOCATION: North Aleutian Shelf
WATER DEPTH: 300'
PRODUCTION RATE: 300000 B/D

LOAD COMBINATION	PLATFORM SHEAR (KIPS) @ MUDLINE			OVERTURNING MOMENT (KIP-FEET) ABOUT		
	X-AXIS	Y-AXIS	VERTICAL	X-AXIS	Y-AXIS	Z-AXIS
1. D+L+B	0	0	103200.	94820.	0	0
2. D+L+B+H+W+C	11110.	10000.	103600.	2140000.	2370000.	92670.
3. D+0.75L+B+E,(Soil B)	4760.	5030.	122000.	1410000.	1310000.	280000.
*4. D+L+B+I+W	8700.	8700.	104000.	2750000.	2660000.	----
*5. D+L+B+S+W	3760.	3760.	104000.	1270000.	1170000.	9800.

REQUIRED PILE GROUP CAPACITIES AT LEG

	A1 and A4	B1 and B4	A2 and A3	B2 and B3
1. D+L+B	33000.	25000.	27000.	18000.
2. D+L+B+H+W+C	34000.	28000.	22000.	17000.
3. D+0.75L+B+E,(Sandy Soil)	29000.	23600.	22000.	17000.
*4. D+L+B+I+W	35000.	29000.	22000.	17000.
*5. D+L+B+S+W	31000.	24000.	21000.	15000.

TABLE 4-26

NATURAL PERIODS AND MODE SHAPES
300' Water Depth
300,000 Barrel per day case

MODE NO.	PERIOD (sec)	DESCRIPTION OF DOMINANT RESPONSE
1	3.262	First lateral Y-mode
2	2.783	First lateral X-mode
3	2.510	First torsional
4	.8816	Complex + second lateral X-mode (inflection between El +65' and El -30')
5	.8721	Complex + second lateral Y-mode (inflection between El +65' and El -30')
6	.8313	Complex + combined Y bending and plan distortion
7	.7907	Complex + second torsional (inflection between El +65' and El -30')
8	.7367	Combined X-bending and torsional
9	.5641	Vertical
10	.4910	Racking + third lateral Y-mode (inflection between El +65' and El -30', and between El -125' and El -230')
11	.4632	Complex
12	.4613	Complex + racking
13	.4212	Complex + plan distortion

TABLE 4-27

JACKET WEIGHT FOR NORTH ALEUTIAN SHELF
(BASE CASE = 300' WATER, 300000 BPD)

Legs (Below Elevation +20')		
Shell	2700	
Diaphragms and conductor guides	640	
Stiffeners	860	
Inner Sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		4500
Vertical Bracing		1600
Skirt Pile Sleeves		1600
Horizontal Bracing		1100
Thickening at joint cans, mill tolerance secondary braces and stiffeners @ 20%		1800
Mudmats		800
Appurtenances		
Launch cradle	650	
J-tubes and risers, sumps	110	
Flooding and grouting systems	90	
Corrosion protection	500	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2100
Total Jacket		13500 tons

TABLE 4-28

PILING AND DECK SUPPORT STRUCTURE
WEIGHTS FOR NORTH ALEUTIAN SHELF
(BASE CASE = 300' WATER, 300000 BPD)

Piling (90 ø 1.5)	
4920' x 0.71 ton/ft	3490
Thick Wall Sections and Stabbing Guides @ 20%	700
Skirt Pile Follower	160
	4350 tons
Deck Support Structures	
Box Girders	2330
Horizontal Bracing	20
Legs (above elevation +20)	600
Temporary Supports	30
	2980 tons

TABLE 4-29

MAXIMUM MUDLINE SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE CAPACITIES FOR VARIOUS WATER DEPTHS AT NORTH ALEUTIAN SHELF (300000 BPD)

	200'	300'	350'
X-shear (kips)	9300.	11110.	11500.
Y-shear (kips)	8400.	10000.	10400.
Vertical force (kips)	100000.	104000.	106000.
Moment about X-axis (kip-feet)	1860000.	2750000.	1770000.
Moment about Y-axis (kip-feet)	3190000.	2660000.	3100000.
Torsion (kip-feet)	62000.	93000.	109000.
Pile Groups A1 & A4 (kips)	32000.	35000.	37000.
Pile Groups B1 and B4 (kips)	26000.	29000.	31000.
Piles B2 and B3 (kips)	17000.	18000.	18000.

TABLE 4-30

PLATFORM STRUCTURE WEIGHT FOR VARIOUS
WATER DEPTHS AT NORTH ALEUTIAN SHELF (300000 BPD)

	200' W.D.	300' W.D.	350' W.D.
Jacket Weight			
Legs	2800	4500	5400
Vertical Bracing	900	1600	1900
Skirt Pile Sleaves	1500	1600	1600
Horizontal Bracing	700	1100	1100
Joint Cans, Secondaries @ 20%	1200	1800	2000
Mudmats	600	800	900
Appurtenances	1600	2100	2300
	9300	13500	15200
Piling (90 ø 1.75)			
Nominal Length and Thickness	2600	3490	3700
Thick Walls and Guides @ 20%	520	700	740
Skirt Pile Follower	110	160	180
	3230	4350	4620 tons
Deck Support Structures	2980	2980	2980
Total	15510 tons	21130 tons	22800 tons

TABLE 4-31

MAXIMUM MUDLINE SHEARS, OVERTURNING MOMENTS AND REQUIRED PILE CAPACITIES FOR VARIOUS WATER DEPTHS FOR 100000 BPD PRODUCTION IN NORTH ALEUTIAN AREA

X-shear (kips)	9000.
Y-shear (kips)	8000.
Vertical force (kips)	76000.
Moment about X-axis (kip-feet)	1760000.
Moment about Y-axis (kip-feet)	1930000.
Torsion (kip-feet)	75000.
Pile Groups A1 & A4 (kips)	25000.
Pile Groups B1 and B4 (kips)	22000.
Piles B2 and B3 (kips)	14000.

TABLE 4-32

JACKET WEIGHT FOR 100000 BPD PRODUCTION
IN NORTH ALEUTIAN AREA (300' WATER)

Legs (Below Elevation +20')		
Shell	2600	
Diaphragms and conductor guides	460	
Stiffeners	540	
Inner Sleeves in double wall section (+20' to -30')	130	
Pile sleeves	170	
		3900
Vertical Bracing		1500
Skirt Pile Sleeves		1700
Horizontal Bracing		1100
Thickening at joint cans, mill tolerance secondary braces and stiffeners @ 20%		1600
Mudmats		700
Appurtenances		
Launch cradle	650	
J-tubes and risers, sumps	110	
Flooding and grouting systems	90	
Corrosion protection	500	
Stairs, boat landing, walkways	80	
Escape pods	650	
		2100
Total Jacket		12600 tons

TABLE 4-33

TONNAGE OF JACKET, DECK SUPPORT STRUCTURE
AND PILING FOR VARIATION IN ENVIRONMENTAL LOADS

	Lower Bound	Design Basis	Upper Bound
St. George	Small decrease (storm, H = 75')	29410 (storm, H = 85')	Small increase (storm, H = 90°)
Navarin	31240 (ice, 80 kips/ft)	32250 (ice, 115 kips/ft)	37980 (ice, 140 kips/ft)
N. Aleutian	Small decrease (storm, H = 65')	20830 (storm, H = 78' ice, 85 kips/ft)	Small increase (ice, 110 kips/ft)

1. Production Rate 300000 BPD for all basins.
2. Water Depth = 450 feet for St. George and Navarin
   = 300 feet for North Aleutian
3. Parentheses indicate the dominant environmental loading.

TABLE OF CONTENTS

SECTION 5.0

Page

LIST OF FIGURES

SECTION 5.0

LIST OF TABLES

SECTION 5.0

5.0 CONCRETE GRAVITY BASE STRUCTURES

5.1 Introduction

In recent years, concrete gravity base structures have been constructed in the hostile waters of the North Sea and have proven to be cost effective in providing a drilling and production structure in deep water. The concrete gravity base structure also has the inherent capability of providing crude oil storage for fields located in remote areas where pipeline construction may prove to be economically prohibitive or require excessive construction time. For these reasons, the concrete gravity base structure was further studied to determine its feasibility for use in the St. George, Navarin and North Aleutian Basins.

5.1.1 Scope of Work

Section 3 describes how the 4 leg concrete structure was qualitatively evaluated and selected for further evaluation.

This analysis of concrete gravity base structures was undertaken to define designs and construction techniques appropriate for the Bering Sea. The analysis considered the specified environmental and operating loads. Geotechnical parameters for both clay and sand sites were utilized to determine safety coefficients for the possible foundation modes of failure.

Global forces were developed for three water depths (300', 450' and 600') using design basis oceanographic criteria from the St. George site. This criteria was used for all basins and is thought to be conservative. Seismic and ice forces were developed separately for each of the basins.

Although site specific forces were not developed for each basin at each water depth, the small differences in the variation of criteria are not believed to affect the accuracy of this feasibility study. The effect of variation in environmental data is further discussed in the sensitivity analysis included in later sections. The emphasis in the preliminary design of the concrete structures was in global load evaluation and foundation integrity. In addition, selected critical locations within the structure were evaluated.

An additional variable that affects the design of concrete gravity base structures is that of crude oil storage capacity. For the two production cases of 100,000 BPD and 300,000 BPD it has been assumed that a 10-day storage capacity would be required. Therefore, desired storage capacities of 1 million to 3 million barrels were included as design parameters in the analysis.

Structures not requiring crude oil storage capacity were also considered. For proper flotation during tow, a minimum amount of closed tankage will be required to provide proper buoyancy. Sufficient base area is required to develop adequate shear resistance against sliding. This is particularly important in the case of clay soils.

Throughout the analysis of the concrete gravity base structures, the assumption has been made that the entire topside dry payload, including drilling and production equipment, will be installed on the concrete base prior to tow out. This requirement, while causing greater demands on the overall stability of the combined structure during tow and installation, greatly reduces the on-site hook-up requirements. The magnitude of the topside dry payload depends on the production rate. The difference in the dry payload is likely to have only a minor effect on the global geotechnical stability of the concrete gravity base structure. However, it may have a significant effect on the floating stability of the structure during tow and installation. In cases of insufficient floating stability it is possible to provide additional stability by means of ballasting the structure to deeper drafts. This can be done using heavy ballast materials such as sand (iron ore), natural sand or concrete. Heavy ballasting has been practiced successfully in several North Sea gravity structures. Alternatively it is possible to enhance the available floating stability by means of temporary buoyancy (Figure 5-1). Where granular ballast has been added to increase stability during tow, it may be desirable to subsequently remove part or all after installation in order to reduce bearing stresses on very soft soils.

5.1.2 Organization of Section

This section of the report presents the design basis, analysis methods, design criteria and preliminary design calculations for the Concrete Base Structure. Section 5.2 discusses general premises and assumptions. Then Sections 5.3, 5.4, and 5.5 present the designs for St. George, Navarin, and North Aleutian Basins.

Evaluation of structural sensitivity to operational or environmental conditions is included in discussion for each basin. Variations of the following parameters are investigated:

Water Depth

Environmental Conditions

Operating Conditions

Soil Conditions

Storage Conditions

Section 5.6 summarizes and discusses the various critical safety coefficients, and Section 5.7 provides insight into structural details and internals of the structure.

Section 5.8 presents construction planning operations, as well as candidate construction sites and qualified contractors for Southern Bering Sea Development.

5.1.3 Structure Configuration

Figure 5-1 shows the proposed structural configuration. Key elements of the base case structure are:

- A cellular base made up of hexagonal cells. The cells could be circular to optimize hydrostatic considerations; however, hexagonal cells minimize framing problems at cell interfaces, and are representative for purposes of this study.

- The base is flared near the mudline to maximize the soil contact area.

- Four tapered shafts which are extensions of four of the base cells extend to the deck level.

- Steel skirts project below the base into stronger soils to enhance sliding and bearing resistance of the structure and minimize the tendency for scour or piping.

5.2 Premises and Assumptions

5.2.1 Materials

The material properties assumed in this study are summarized in Table 5-1.

5.2.2 Codes Utilized

The following documents provided guidance in developing the various analysis procedures and design criteria:

1. American Concrete Institute, 1978. "Guide for the Design and Construction of Fixed Offshore Concrete Structures." ACI Report 357R-78.

2. American Petroleum Institute, 1982. "API Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms." API RP 2A 13th Edition.

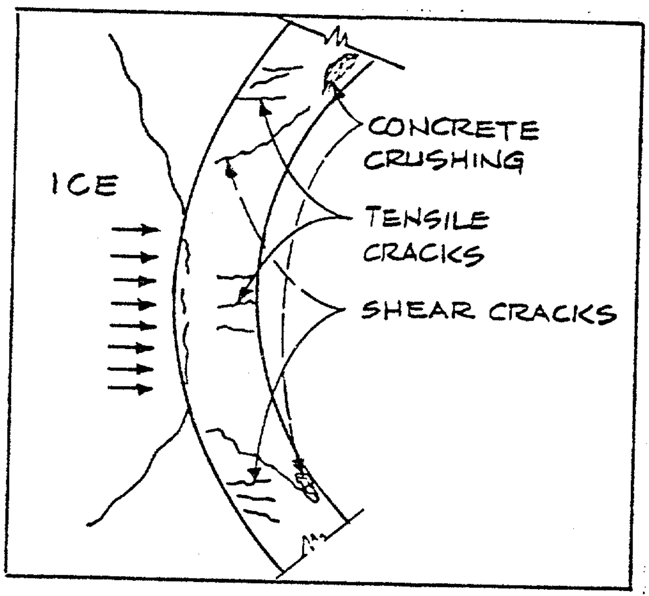
3. American Petroleum Institute, 1982. "API Bulletin on Planning, Designing and Constructing Fixed Offshore Structures in Ice Environments." Bul 2N 1st Edition.

4. Det Norske Veritas, 1977. "Rules for the Design, Construction and Inspection of Offshore Structures."

5. Federation Internationale de la Precontrainte, 1977. "Recommendations for the Design and Construction of Concrete Sea Structures." 3rd Edition.

5.2.3 Structural Design Considerations

5.2.3.1 Local Concentrated Ice Loads



FAILURE MODES

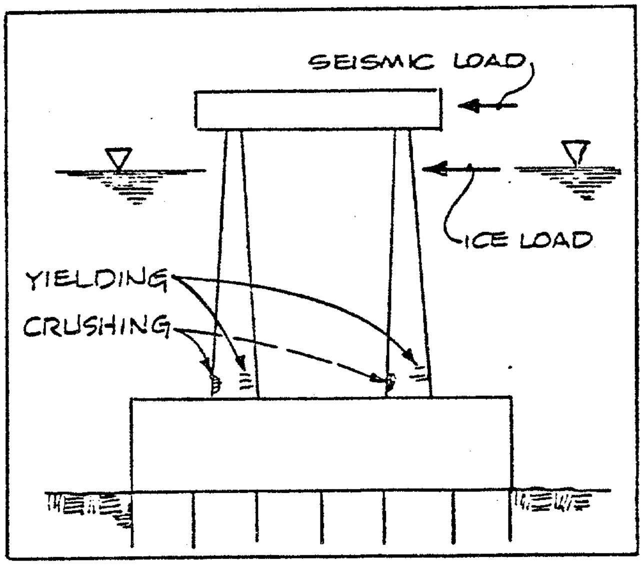
This failure mode is often referred to also as punching shear failure. High concentrated ice loads acting on the shafts may cause combined flexural crushing and shear of the concrete. The failure mechanism is a progressive process involving the formation of flexural cracks, internal arching, shear cracks, and concrete crushing. The formation of cracks results in stress redistribution, leading to ultimate failure by crushing of the concrete or punching shear. Potential solutions can be provided as follows:

a. Increased wall thickness at the contact zone.

b. Increase concrete strength.

c. Triaxial confinement of the concrete. The latter is most effectively achieved by means of closely spaced through-wall stirrups or through-wall prestressing, as by rock bolts.

5.2.3.2 Failure at the Base of the Shafts



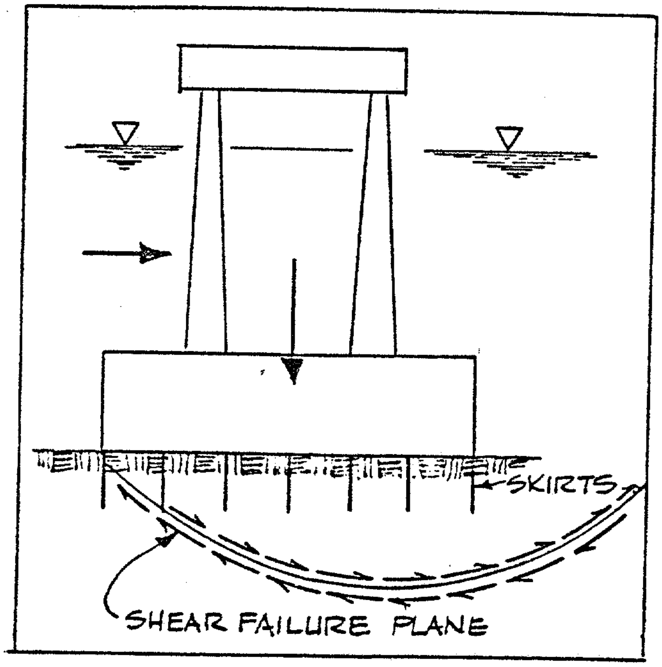
High magnitude seismic or ice lateral loads generated at the deck or the waterline levels, respectively, may result in excessively high bending moments at the base of the shafts. Thus, exceedance of both the steel tension capacity and/or the concrete compression capacity may occur. These overstresses may result in steel yielding and concrete cracking and crushing. In designing for the ultimate limit state, ductile steel yielding is normally preferred over brittle concrete crushing.

The failure mode can be prevented by increasing the wall thickness as well as tapering the shafts to increase the moment resisting lever arm. Local confinement of the concrete by means of stirrups or transverse prestressing is advantageous in increasing section ductility.

5.2.4 Foundation Considerations

Ice, waves and earthquakes are the most important environmental loads in determining the foundation stability of concrete gravity base structures for the Bering Sea. These loads need to be considered together with the buoyant weight of the structures. In order to determine the critical design conditions, it is necessary to evaluate each of the most probable loading combinations for several potential modes of failure. The predominant failure modes are as follows:

.2.4.1 Global Base Shear



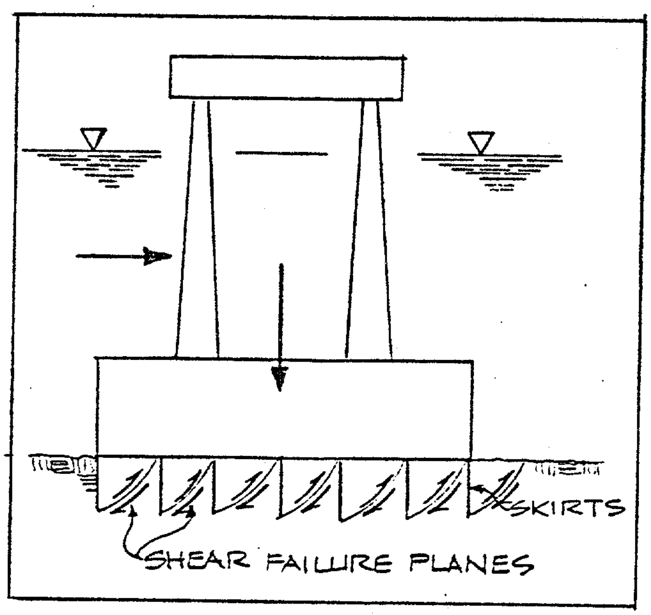
Exceedance of the shear resistance capacity of the foundation soil will cause the structure to slide. This failure mode can be prevented in the following ways:

a. In granular soils (sands) - use of additional surcharge weight.

b. In cohesive soils (clays) - extension of the base area.

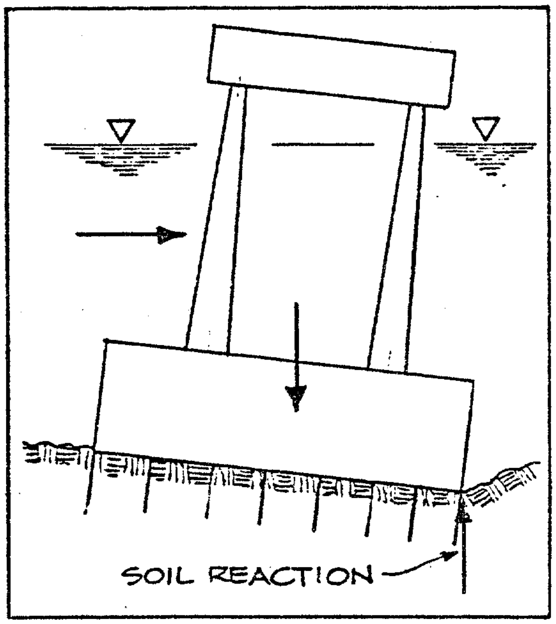
In both cases the utilization of skirts under the base will help to prevent sliding failure. This is because the potential shear failure plane will be forced deeper into stronger soil formations. Furthermore, the shear area will be increased due to the concavity of the potential failure plane.

5.2.4.2 Shear around Skirt Tips



This may happen due to the development of a series of local shear failure planes in the weak near surface soils. The use of additional skirts to reduce the inter-skirt spans is useful in preventing this failure because the potential shear failure planes are again forced to greater depths. Furthermore, the skirts should subdivide the total base area into individual segments. This partitioning will help to prevent the local shear failure planes from extending throughout the entire base area.

5.2.4.3 Overturning



A concrete gravity base structure is subject to high lateral loads, resulting in large overturning moments. These are counteracted by the structure's buoyant weight together with other vertical force components acting simultaneously with the maximum overturning moments.

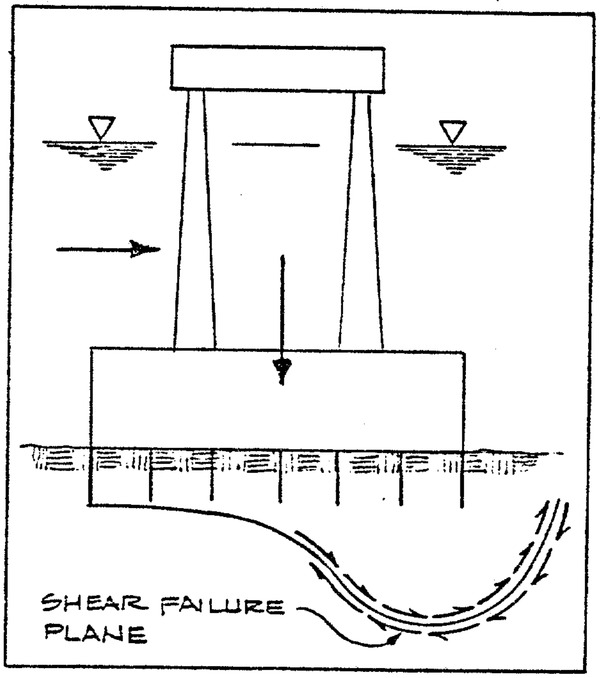
The overturning resistance capacity can be increased using the following methods:

1. Additional surcharge weight.

2. Extension of the base area.

The choice of the most suitable technique may vary depending upon the specific conditions at each location. For example, additional surcharge weight is recommended for granular (sand) soil sites. Conversely, extending the base area is more suitable to cohesive (clay) soils. Separation of the base area is a potential solution in cases of deep water and soft soils sites. In any event, it is necessary to ensure that the allowable bearing stresses are not exceeded.

.2.4.4 Bearing



Bearing failure occurs when a shear plane develops due to excessive bearing stresses in the soil underneath the structure. The area contained by the shear failure plane may include the entire base or portions of it. This mode of failure is most likely to occur in weak undrained cohesive soils.

Long skirts are useful in preventing bearing failures by pushing the shear plane deeper into stronger soil strata. Another possible solution is to reduce the structure's buoyant weight. This may require some modifications in the structural design such as the use of lighter materials or provision of buoyancy by means of dewatered compartments within the structure.

.2.4.5 Evaluation Procedures and Criteria

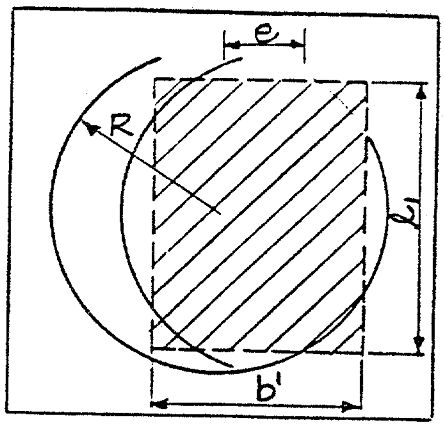
The procedures used for the geotechnical stability are summarized below:

Soil Type	Failure Mode	Procedure	Criterion (Per API RP 2A)
Cohesive	Sliding	AC/F_H	1.5
	Overturning	F_VR/M	2.0
	Bearing	qA'/F_V	2.0
Granular	Sliding	$F_V\tan \phi/F_H$	1.5
	Overturning	F_VR/M	2.0
	Bearing	qA'/F_V	2.0

where

- A = Gross foundation area.
- A' = Effective foundation area.
- C = Undrained shear strength.
- q = Allowable bearing pressure.
- ϕ = Angle of internal friction.
- R = Characteristic foundation dimension.
- F_H = Horizontal force.
- F_V = Vertical force.
- M = Overturning moment.

The ratios obtained by using the procedures specified in the table above are the actual safety coefficients. These coefficients should in all cases be equal to or larger than the corresponding criteria. In other words, the criterion indicates the minimum requirement for each case. The criteria selected here are based on the American Petroleum Institute recommendations. It should be noted that other requirements, such as material and load coefficients, are also included in the specified criteria.



EFFECTIVE FOUNDATION AREA
CIRCULAR GEOMETRY

- EFFECTIVE FOUNDATION (A')

$$A' = \pi R^2 - 2\left[e(R^2 - e^2)^{1/2} + R^2 \sin^{-1}\left(\frac{e}{R}\right)\right]$$

$$\ell' = A'\left[(R+e)/(R-e)\right]^{1/2}$$

$$b' = \ell'\left[(R-e)/(R+e)\right]^{1/2}$$

NOTE: THE HEXAGON BASE IS APPROXIMATED USING CIRCULAR GEOMETRY HAVING THE SAME BASE AREA.

The characteristic foundation dimension $(\hat{R})$ is equal to one half of the foundation diameter for circular foundations. (See above sketch.) For other foundation geometries it can be taken as one half of the distance between the furthest two points parallel to the direction of the applied load.

The effective foundation area (A') is used in the evaluation of the bearing stabililty. This approach is recommended both by the American Petroleum Institute (API) and by the Norwegian Classification Society (Det Norske Veritas). The simultaneous exposure of the structure to lateral and vertical loads results in eccentric foundation reaction and thus nonuniform pressure distribution. The effective foundation describes an area subject to an equivalent uniform pressure distribution. The total foundation reaction in both cases is the same.

5.2.5 Loads

Forces acting on offshore structures are due to the combination of different environmental and operating conditions. The various load combinations are normally examined for compliance with the requirements of two limit states, Serviceability and Ultimate. In this study only the Ultimate Limit State loads are used, since this controls the overall structural dimensions. The Serviceability Limit State usually determines the amount of reinforcing steel required and an allowance has been made for the quantity, based on previous experience. Since concrete gravity base structures are insensitive to changes in the operating loads, the payload capacity is kept at its maximum level at all times. It is also assumed that the different environmental extreme loads such as oceanographic, seismic and ice, act independently and nonconcurrently, a procedure which is in accord with the recommendations and practice.

5.2.5.1 Oceanographic Loads

Wave loads have a dominant effect on the global geotechnical stability of concrete gravity base platforms. Hence other loads such as winds and currents were not considered in the evaluation presented here. A deterministic design wave approach was used in this study. Approximate wave loads were initially obtained using a closed form diffraction theory solution by Gran (1). More accurate loads were subsequently obtained using a numerical diffraction theory solution by Garrison (2).

---

(1) Gran, S. 1973, "Wave Forces on Submerged Cylinders," Proc. OTC No. 1817, Houston, Texas, May.

(2) Garrison, C. J., Chow, P. Y., 1972, "Wave Forces on Submerged Bodies," Journal of Waterways, Harbors, and Coastal Engineering Division, August.

2.5.2 Seismic

Seismic forces may become a dominant consideration in the design of gravity base structures. This depends upon the seismicity of each site as well as the structures' configurations and dynamic characteristics. For preliminary design purposes the global lateral forces were estimated assuming rigid body motions. Following this assumption the seismic forces were found by multiplying the masses of the structures, including those of contained liquids, times the anticipated response accelerations. The latter were obtained from the appropriate acceleration response spectra based on the assumed fundamental vibration frequencies. Added mass coefficents were based on the analytical work of Garrison confirmed by the experimental work of Byrd (2). In all cases the global seismic forces were applied at the centers of gravity resulting in overturning moments about the mud line.

Internal structural damping is important only to the elements of the structure, such as the shaft-base connection. Hydrodynamic damping is also insignificant for the structural configurations and water depth considered in this report. Radiation damping in the soil, however, is of major importance.

Such a large mass bearing on the soil compels the foundation to respond to a large degree with the structure. This interaction of underlying soil with the structure has been given detailed analytical treatment in connection with the similar problem for nuclear reactor power plants. In particular, the development of a 3-D finite element program by Lysmer appears to offer a more valid method for analysis.

The net result is that the high frequency energy is both filtered and dampened by the soil, so that the response of the structure is controlled by velocity rather than acceleration. Thus, the peak spectral acceleration of a particular earthquake has little or no effect on the response of the gravity base structure.

Very detailed evaluations have recently been performed for one of the major gravity base structures in the North Sea for a seismic event developing a similar spectral velocity to that given for the St. George Basin. The forces and moments should thus be comparable, at least in a general sense. The values presented in this report are in reasonable agreement.

Experience from detailed seismic evaluation of a concrete gravity base structure in the North Sea indicates that earthquake can govern the reinforcing and prestressing in the shafts but is not critical elsewhere in the concrete substructure.

Vertical accelerations have not been considered in the present evaluation but should be taken into account in a specific design.

It has been found from the North Sea evaluation that both horizontal and vertical motions do affect the design of the deck and the support of the equipment.

In view of the fact that the Aleutian chain is one of the most seismically active regions in the world, and the fact that it is a plate margin, rather than an interplate zone, we recommend that a more in-depth study be made of the earth motion characteristics, so as to more accurately define the design parameters, not only of horizontal and vertical accelerations but particularly of velocity, displacement and duration applicable to each basin.

5.2.5.3 Ice

Global ice loads are generated by a variety of ice formations such as ridges, floes and rafted ice. In the case of the selected four-legged concept ice may get "jammed" among the shafts, resulting in higher loads due to an arching action. This possibility was considered in all ice force calculations. Furthermore, the failure loads were calculated assuming that the ice fails in crushing. In all cases the global ice loads were applied at the water line, resulting in overturning moments about the mud line.

2.5.4 Loading Combination

To account for various potentially critical loading conditions, the following combinations were considered:

Case I = $F_B + F_{WH} + M_W$

Case II = $F_B - F_{WV}$

Case III = $F_B + F_{WH} + 0.5\ F_{WV} + M_W$

Case IV = $F_B + 0.5\ F_{WH} + F_{WV} + 0.5\ M_W$

Case V = $F_B + F_{SH} + M_S$

Case VI = $F_B + F_{IH} + M_I$

where:

F_B = Maximum buoyant weight.

F_{WH} = Maximum wave horizontal force.

F_{WV} = Maximum wave vertical force.

M_W = Maximum wave overturning moment.

F_{SH} = Maximum seismic horizontal force.

M_S = Maximum seismic overturning moment.

F_{IH} = Maximum ice horizontal force.

M_I = Maximum ice oveturning moment.

Notes:

1. Load cases I and II correspond to a linear wave condition.
2. Load cases III and IV account for potential nonlinearities.
3. Load case V represents the maximum seismic loading.
4. Load case VI represents the maximum ice loading.

5.2.6 Auxiliary Systems

Pipeline risers, ballast control equipment, disposal caissons, diver and emergency egress are all systems which should be incorporated into the design of the concrete structures. Their location should be carefully planned so as not interfere with the primary load-carrying systems of the structures.

The spaces within the concrete caisson are highly suitable and adaptable for bulk storage of various supplies and materials, such as:

Fresh water

Drilling mud

Cement

Diesel Oil

Drill cuttings disposal

The storage space allocation for the above components should be done considering the primary requirements for ballast and crude oil storage.

5.3 St. George Basin

5.3.1 Base Case

The base case structure for the St. George Basin is to be founded on cohesive (clay) soil in 450 feet of water. The structure is designed to accommodate a production rate of 300,000 BPD and provide a storage capacity of at least 1 million barrels. A general layout of the proposed structural configuration is shown in Figure 5-1. The analytical results given below are primarily to evaluate suitability of the base case design to resist global loads under the specified environmental, operational and geotechnical criteria.

5.3.1.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip(1)
Storage	=	1×10^6 bbl

(1) 91,800 kip rounded up.

B. Environmental

Oceanographic - wave height = 85 ft.

wave period = 15 sec.

Seismic - peak ground acceleration = 0.125g(2)

- velocity = 5 inc/sec

Ice - rafted ice = 85 kip/ft

C. Geotechnical

Clay - undrained shear strength = 1.0 ksf(3)

5.3.1.2 Results of Analysis

In conducting the analysis presented here, no attempt was made to optimize the structural geometry or weight characteristics. The initial configuration was assumed using past experience and judgment. The mininum ballast requirements was determined such that the submerged weight of the structure will be sufficient to resist the maximum potential wave uplift. Furthermore, no attempt was made to determine the optimum ballast distribution. It was assumed that the ballast will be provided using sea water in part of the base compartments such that the center of gravity of the ballast will be kept at mid-height of the base.

(2) The specified acceleration is the peak ground acceleration; however, the seismic forces were calculated based on the design basis response spectrum. Based on an anticipated fundamental response period of 2 sec., the estimated spectral acceleration from the response spectrum is less than 0.03g, but was rounded up to .05 g to account for possible underconservatism in the static analysis method which is based on the fundamental mode of vibration.

(3) At the skirt tip 15 feet below the mudline.

A. Operating

Air weight of GBS plus topside = 957,000 kip

Minimum ballast required = 473,000 kip

Total air weight = 1,430,000 kip

B. Environmental

Oceanographic - horizontal = 9.2×10^4 kip

vertical = 7.9×10^4 kip

overturning = 8.7×10^6 kip-ft

Seismic - horizontal = 1.0×10^5 kip

overturning = 9.7×10^6 kip-ft

5.3.1.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-1.

	Coefficient		
Case Criteria	Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.6	3.5	5.7
II	---	---	31.8
III	2.6	4.8	5.2
IV	5.2	12.0	5.3
V	2.3	3.1	6.1
VI	11.1	3.0	7.2

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding is controlled by the maximum seismic loading, although design wave loading is only slight less. Overturning is controlled by the maximum ice loading. Bearing is controlled by the wave loading. It can also be concluded that the lower bound (weakest) cohesive soils (.5 ksf @ 15 ft.) would not be suitable foundation materials.

5.3.1.4 Material Take Off

1.	Base slab	54409 CY
	Enlargement at base-skirt connection	1000 CY
2.	Interior cell walls	69924 CY
	Enlargement at base-wall connection	1000 CY
3.	Exterior cell walls	14107 CY
	Exterior cell walls	9069 CY
4.	Roof of cells	18734 CY
5.	Towers	41334 CY
6.	Apron	7000 CY
7.	Ring beam	1000 CY
8.	Miscellaneous	10000 CY
	Total concrete in base case	227577 CY

5.3.2 Water Depth Variations

In order to accommodate changes in water depth for 300 and 600 feet, the following design changes were made from the base case structure.

300' W. D.: The base features were kept the same. The shafts were shortened 150', and the shafts were specified at a constant diameter of 40 feet instead of the tapered shafts of the base case. No other design changes were made (Figure 5-2).

600' W.D. - In order to maintain acceptable moments at the base of the shafts, the base height was increased to 200'. This has the additional benefit of permitting possible oil storage capacity of up to 3 million barrels. The shafts were lengthened by 50 feet by extending the 75' diameter region at the base of the shafts (Figure 5-3).

With these design changes, the global loads were evaluated for the design conditions detailed below. The primary objectives of this work were to establish water depth limits of this design under the specified environmental, operational and geotechnical criteria. No attempt was made to optimize the design for the different water depths.

5.3.2.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics.

		300'	600'
A. Operating			
Payload	=	100,000 kip	100000 kip
Storage	=	1×10^6 bbl	3×10^6 bbl

B. Environmental

Oceanographic - wave height	=	75 ft.(1)	85 ft.
wave period	=	14 sec.	15 sec.
Seismic - peak ground acceleration	=	0.125g(2)	0.125 g(2)
- velocity		5 in/sec	5 in/sec
Ice - rafted ice	=	85 kip/ft	85 kip/ft

C. Geotechnical

Clay - undrained shear strength	=	1.0 ksf(3)	1.0 ksf(3)

5.3.2.2 Results of Analysis

See discussion in Section 5.3.1.2.

A. Operating

Air weight of GBS plus topside	=	858000 kip	1369000 kip
Minimum ballast required	=	562000 kip	1150000 kip
Total air weight	=	1420000 kip	2519000 kip

(1) A wave height of 75' was used as opposed to the 85' used for the base case. The results of the analysis indicate ample stability for the 300' case; however, the 85' wave case was not directly examined.

(2) See note (2) in Section 5.3.1.1.

(3) At the skirt's tip 15 feet below the mudline.

			300'	600'

B. Environmental

Oceanographic	- horizontal	=	1.1×10^5 kip	1.6×10^5 kip
	vertical	=	1.1×10^5 kip	5.5×10^4 kip
	overturning	=	14.4×10^6 kip-ft	25.3×10^6 kip-ft
Seismic	- horizontal	=	1.0×10^5 kip	1.8×10^5 kip
	overturning	=	8.7×10^6 kip-ft	24.5×10^6 kip-ft
Ice	- horizontal	=	2.0×10^4 kip	2.0×10^4 kip
	overturning	=	6.0×10^6 kip-ft	12.0×10^6 kip-ft

It is interesting to note that the oceanographic forces in both of the above cases are larger than the corresponding forces for the base case. This can be explained in the case of the shallow structure (W.D. = 300 feet) due to the relative increase in the wave energy per unit of water depth. The maximum wave considered in this case is just slightly smaller than the base case maximum wave while the water depth ratio is equal to two thirds. The increased oceanographic loads in the case of the deeper structure (W.D. = 600 feet) are due to the larger size of the base caisson. This, together with the higher elevation of the topside mass above the mudline are also responsible for the increase in the magnitude of the seismic loads.

The ice load is applied at the mean water line and therefore, the resulting overturning moment is a direct function of the water line elevation above the mud line.

C. Loading Summary

Based on the relative magnitudes of the environmental loads, it is concluded that the maximum ice loading does not represent a critical design condition and thus is not considered in subsequent calculations for these cases.

	Load					
	Horizontal (kip)		Vertical (kip)		Overturning (kip-ft)	
Case	300 ft.	600 ft.	300 ft.	600 ft.	300 ft.	600 ft.
I	1.1×10^5	1.6×10^5	1.4×10^5	2.2×10^5	14.4×10^6	25.3×10^6
II	------		3.3×10^4	1.6×10^5	------	------
III	1.1×10^5	1.6×10^5	2.0×10^5	2.5×10^5	14.4×10^6	25.3×10^6
IV	5.5×10^4	8.0×10^4	2.5×10^5	2.8×10^5	7.2×10^6	12.6×10^6
V	1.0×10^5	1.8×10^5	1.4×10^5	2.2×10^5	8.7×10^6	24.5×10^6

5.3.2.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structures shown in Figures 5-2 and 5-3.

	Coefficient					
	Sliding		Overturning		Bearing	
	300 ft.	600 ft.	300 ft.	600 ft.	300 ft.	600 ft.
Criteria	1.5	1.5	2.0	2.0	2.0	2.0
I	2.2	1.5	2.7	2.4	2.3	1.4
II	---	---	---	---	45.6	9.2
III	2.2	1.5	3.8	2.7	2.9	1.3
IV	4.4	2.9	9.7	6.0	4.4	3.3
V	2.5	1.3	4.5	2.5	5.2	1.4

Based on the above results, it is observed that the shallower structure (W.D. = 300 ft) is stable in all modes. In the case of the deeper (W.D. = 600 ft) structure, the minimum sliding and bearing requirements are not satisfied. The sliding is controlled by the maximum seismic load (although the design wave load is only slightly less); whereas, the bearing is controlled by the maxima of both the wave and the seismic loads. As indicated in the previous section, both problems are primarily due to the increase in the base height. The results obtained indicate a limiting condition for the selected structural configuration given for the specific water depth, environmental loads, and soil properties. Other alternatives such as the tripod gravity base structure (Figure 5-4) will probably prove more suitable for the design conditions of deep water and soft soil. The tripod serves to reduce the change in bearing pressure due to moment, as well as providing reduced uplift. This provides a reduction in the net static weight on the soil. It should be noted here that the selected structural alternative will fully satisfy the prescribed geotechnical stability criteria in the case of the upper bound clay soils (i.e., where the undrained shear strength equals 2 ksf).

5.3.2.4 Material Take Off

St. George - Water Depth Variation

A. W. D. = 300 and 600 feet	300'	600'
1. Base slab	54409 CY	54409 CY
Enlargement at base-skirt connection	1000 CY	1000 CY
2. Interior cell walls	69924 CY	139848 CY
Enlargement at base-wall connection	1000 CY	1000 CY
3. Exterior cell walls	14107 CY	28214 CY
Exterior cell walls	9069 CY	18138 CY
4. Roof of cells	18734 CY	18734 CY
5. Towers	17760 CY	46314 CY
6. Apron	7000 CY	7000 CY
7. Ring beam	1000 CY	1000 CY
8. Miscellaneous	10000 CY	10000 CY
Total Concrete in Structure	204003 CY	325657 CY

5.3.3 Environmental Conditions Variations

The following work was done in order to evaluate the sensitivity of the base case structure to changes in the environmental conditions. As discussed in Section 5.3.1, the base case global loading was controlled by seismic loading for sliding, ice loads for overturning and wave loads for bearing. Of these, the sliding and overturning modes were the only cases even approaching the limiting criteria. For purposes of this phase of the sensitivity evaluation, only the ice load was increased from 85 k/ft to 110 k/ft. The seismic spectral acceleration was not increased because, as explained in Section 5.3.1.1, the initial assumption was believed to be conservative already. All other conditions were also kept the same as in the base case. The structural configuration was not modified for the changes in the ice conditions. Only the stability against global loading was reevaluated.

5.3.3.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. <u>Operating</u>

Payload	=	100,000 kip
Storage	=	1×10^6 bbl

B. <u>Environmental</u>

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.125g(1
- velocity	=	.125g(1)
Ice - rafted ice	=	110 kip/ft

C. <u>Geotechnical</u>

Clay - undrained shear strength = 1.0 ksf

5.3.3.2 Results of Analysis

A. <u>Operating</u>

Air weight of GBS plus topside	=	957000 kip
Minimum ballast required	=	473000 kip
Total air weight	=	1430000 kip

B. <u>Environmental</u>

Oceanographic	- horizontal	=	9.2×10^4 kip
	vertical	=	7.9×10^4 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	1.0×10^5 kip
	overturning	=	7.2×10^6 kip-ft
Ice	- horizontal	=	2.6×10^4 kip
	overturning	=	12.7×10^6 kip-ft

(1) See note (2) in Section 5.3.1.1.

The changes in the horizontal and overturning ice loads as compared to the base case are directly proportional to the increase in the ice strength.

C. Loading Summary

Case	Load Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	9.2×10^4	1.1×10^5	8.7×10^6
II	----	5.1×10^4	----
III	9.2×10^4	1.5×10^5	8.7×10^6
IV	4.6×10^4	1.9×10^5	4.4×10^6
V	1.0×10^5	1.1×10^5	7.2×10^6
VI	2.6×10^4	1.1×10^5	12.7×10^6

5.3.3.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5.5.

	Coefficient		
Case Criteria	Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.6	3.5	5.7
II	---	---	31.8
III	2.6	4.8	5.2
IV	5.2	12.0	5.3
V	2.3	4.2	6.1
VI	9.2	2.4	5.7

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding is still controlled by the maximum seismic loading. Overturning is controlled by the maximum ice loading and bearing is still controlled by the wave loading.

5.3.3.4 Material Take Off

Same as base case.

5.3.4 Operating Conditions Variation

Reducing the crude oil production rate from 300000 BPD to 100000 BPD, thereby reducing the payload, has only a marginal effect on the global weight and geotechnical stability of concrete gravity base structures. Hence this variation was not considered as a separate design case. Nevertheless, changes in the payload capacity may have a significant effect during the transportation and installation phases. Both the motion response and the hydrostatic stability during tow and installation strongly depend upon the magnitude and elevation of the topside payload.

5.3.5 Soil Conditions Variation

Changes in the soil conditions at the platform location will have a profound effect on the suitability of the base case design to satisfy the prescribed criteria. As discussed in Sections 5.3.1 and 5.3.2, the base case cohesive soils were found to be suitable for the postulated criteria except in the deep water range. As indicated earlier (Section 5.3.2.3), based on previous calculations the geotechnical criteria is expected to be satisfied in the case of upper bound cohesive soils. The primary objective of this phase of the sensitivity evaluation was to investigate whether granular soils near the postulated lower bound conditions imposed any additional global loading limitations.

The design of the structure was the same as in the base case, however, the ballast was increased to enhance behavior on the granular soils. It should be noted here that in granular soils, unlike cohesive soils, the sliding resistance capacity of the foundation can be increased when additional ballast is used.

However, large ballast amounts may also have adverse effects on the geotechnical stability due to the exceedance of the allowable bearing pressures as well as increased horizontal and overturning seismic loads. Hence, the decision concerning the actual amount of ballast to be used must consider all the stability requirements simultaneously.

The stiffer granular soils affect the seismic response by decreasing the fundamental period of response and also decreasing the damping.

5.3.5.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip
Storage	=	1×10^6 bbl

B. Environmental

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.125g(2)
- velocity	=	5 in/sec
Ice - rafted ice	=	85 kip/ft

C. Geotechnical

Sand - internal friction angle	=	30°(1)

(1) The internal friction angle was conservatively chosen to represent sands midway between lower bound and design basis at the skirt tips, 15 feet below the base line.

(2) See note (2) in Section 5.3.1.1.

5.3.5.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	957000 kip
Minimum ballast required	=	803000 kip
Total air weight	=	1760000 kip

B. Environmental

Oceanographic	- horizontal	=	9.2×10^4 kip
	vertical	=	7.9×10^4 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	1.3×10^5 kip
	overturning	=	11.3×10^6 kip-ft
Ice	- horizontal	=	2.0×10^4 kip
	overturning	=	9.8×10^6 kip-ft

The increased ballast and changed soil-structure interaction affect only the seismic loads which are somewhat higher than in the base case.

C. Loading Combinations

Case	Load: Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	9.2×10^4	4.4×10^5	8.7×10^6
II		3.4×10^4	
III	9.2×10^4	4.8×10^5	8.7×10^6
IV	4.6×10^4	5.2×10^5	4.4×10^6
V	1.2×10^5	4.4×10^5	10.3×10^6
VI	2.0×10^4	4.4×10^5	9.8×10^6

5.3.5.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-6.

Case Criteria	Coefficient: Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.8	14.0	32.8
II	---	---	48.5
III	3.0	15.2	30.2
IV	6.5	33.3	29.6
V	2.0	10.7	30.3
VI	11.8	12.0	31.8

5.3.5.4 Material Take Off

Same as base case.

5.3.6 Storage Conditions Variation

In this section the storage requirements of the base case structure (Figure 5-1) are varied. The base case structure was modified to store 3 million barrels of crude oil (see Figure 5-5). This was accomplished by increasing the base height to 200' with the same base plan dimensions. Storage below 1 million barrels would result in savings in the mechanical equipment, but would not reduce the base dimension or concrete volume beyond the base case due to minimum ballast and structural integrity requirements.

5.3.6.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip
Storage	=	3×10^6 bbl

B. Environmental

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.125g[1]
- velocity	=	5/in/sec
Ice - rafted ice	=	85 kip/ft

C. Geotechnical

Clay - undrained shear strength = 1.0 ksf

5.3.6.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	1307000 kip
Minimum ballast required	=	1003000 kip
Total air weight	=	2310000 kip

B. Environmental

Oceanographic	- horizontal	=	1.5×10^5 kip
	vertical	=	1.0×10^5 kip
	overturning	=	7.2×10^6 kip-ft
Seismic	- horizontal	=	1.7×10^5 kip
	overturning	=	21.7×10^6 kip-ft
Ice	- horizontal	=	2.0×10^4 kip
	overturning	=	9.8×10^6 kip-ft

C. Loading Summary

Based on the relative magnitudes of the environmental loads it is concluded that the maximum ice loading does not represent a critical design condition and thus is not considered in subsequent calculations for this case.

(1) See note 2 in Section 5.3.1.1.

Case	Load		
	Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	1.5×10^5	2.2×10^5	7.2×10^6
II	-----	1.2×10^5	-----
III	1.5×10^5	2.7×10^5	7.2×10^6
IV	0.8×10^5	3.2×10^5	3.6×10^6
V	1.7×10^5	2.2×10^5	21.7×10^6

5.3.6.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-5.

Case	Coefficient		
Criteria	Sliding 1.5	Overturning 2.0	Bearing 2.0
I	1.6	8.4	2.4
II	---	---	11.9
III	1.6	10.3	2.0
IV	3.1	24.2	3.4
V	1.5	2.8	1.6

From the above results it is observed that the structure does not satisfy the minimum stability requirements in bearing. It is further noted that all modes of failure are controlled by the maximum seismic loading. If the selected structure is to be founded on the upper bound soils, then the mininum bearing requirement can be easily satisfied. It should be further noted that since the geotechnical stability in this case was determined by the seismic load which was conservatively assumed, it may be beneficial to further investigate this subject.

5.3.6.4 Material Take Off

St. George - Storage Conditions Variation (3 Million Barrels)

1.	Base slab	54409 CY
	Enlargement at base-skirt connection	1000 CY
2.	Interior cell walls	139848 CY
	Enlargement at base-wall connection	1000 CY
3.	Exterior cell walls	28214 CY
	Exterior cell walls	18138 CY
4.	Roof of cells	18734 CY
5.	Towers	31374 CY
6.	Apron	7000 CY
7.	Ring beam	1000 CY
8.	Miscellaneous	10000 CY
		310717 CY

5.4 Navarin Basin

The various structures and conditions examined in the Navarin Basin are the same as in the St. George Basin. Adjustments were made only for the different levels of the seismic and ice loads.

5.4.1 Base Case

The base case structure for the Navarin Basin is to be founded on cohesive (clay) soil in 450 feet of water. The structure is designed to accommodate a production rate of 300,000 BPD and provide a storage capacity of 1 million barrels. A general layout of the proposed structural configuration is shown in Figure 5-1. The analytical results given below are primarily to evaluate the suitability of the base case design to resist global loads under the specified environmental, operational and geotechnical criteria.

5.4.1.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip[(1)]
Storage	=	1×10^6 bbl

(1) 91,900 kip rounded up.

B. Environmental

Oceanographic - wave height = 85 ft.

wave period = 15 sec.

Seismic - peak ground acceleration = 0.05g[(2)]

- velocity = 2 inc/sec

Ice - rafted ice = 110 kip/ft

C. Geotechnical

Clay - undrained shear strength = 1.0 ksf[(3)]

5.4.1.2 Results of Analysis

Since the base case structure for the Navarin Basin is identical to the base case structure for the Saint George Basin, no attempt was made to optimize the structural design.

A. Operating

Air weight of GBS plus topside = 957000 kip

Minimum ballast required = 473000 kip

Total air weight = 1430000 kip

(2) The specified acceleration is the peak ground acceleration; however, the seismic forces were calculated based on the design basis response spectrum. The anticipated fundamental response period is 2 sec, which is beyond the limits of the proposed acceleration response spectrum. Hence, the lower bound acceleration of 0.01 g was conservatively selected for the Navarin Basin.

(3) At the skirt tips 15 feet below the mudline.

B. Environmental

Oceanographic	- horizontal	=	9.2×10^4 kip
	vertical	=	7.9×10^4 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	2.0×10^4 kip
	overturning	=	1.9×10^6 kip-ft
Ice	- horizontal	=	2.8×10^4 kip
	overturning	=	12.6×10^6 kip-ft

It should be noted that both the seismic and the ice loads in this case were prorated based on the corresponding forces for the Saint George Basin in direct proportion to the spectral acelerations ratio and the unconfined compressive strengths ratio, respectively.

C. Loading Summary

Case	Load Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	9.2×10^4	1.1×10^5	8.7×10^6
II		3.1×10^4	
III	9.2×10^4	1.5×10^5	8.7×10^6
IV	4.6×10^4	1.9×10^5	4.4×10^6
V	2.0×10^4	1.1×10^5	1.9×10^6
VI	2.8×10^4	1.1×10^5	12.6×10^6

5.4.1.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-1.

Case Criteria	Coefficient Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.6	3.5	5.7
II	---	---	31.8
III	2.6	4.8	5.2
IV	5.2	12.0	5.3
V	11.9	15.9	12.0
VI	18.5	2.4	5.7

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding and bearing are controlled by the maximum wave loading. Overturning is controlled by the maximum ice loading. It can also be concluded that the lower bound cohesive soils (.5 ksf at 15 feet) would not be suitable foundation material due to inadequate shear resistance to sliding.

5.4.1.4 Material Take Off

Total Concrete Volume	227577 CY

Concrete volume is the same as that found in Section 5.3.1.4 - St. George Basin; Base Case.

5.4.2 Water Depth Variation

Variations in the water depth are examined for the conditions of 300 feet and 600 feet (Figures 5-2 and 5-3). For a discussion of other design changes included in this section, see Section 5.3.2.

5.4.2.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics.

		300'	600'
A. <u>Operating</u>			
Payload	=	100,000 kip	100,000 kip
Storage	=	1×10^6 bbl	3×10^6 bbl
B. <u>Environmental</u>			
Oceanographic - wave height	=	75 ft.(1)	85 ft.
wave period	=	14 sec.	15 sec.
Seismic - peak ground acceleration	=	0.05g	0.05g(2)
- velocity	=	2 in/sec	2 in/sec
Ice - rafted ice	=	110 kip/ft	110 kip/ft

(1) A wave height of 75 feet was used as opposed to the 85 feet used for the base case. The results of the analysis idnicate ample stability for the 300 foot case, however the 85 foot wave is not directly examined.

(2) See note (2) in Section 5.4.1.1.

.4.2.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	858000 kip	1369000 kip
Minimum ballast required	=	562000 kip	1150000 kip
Total air weight	=	1420000 kip	2519000 kip

B. Environmental

			300'	600'
Oceanographic	- horizontal	=	1.1×10^5 kip	1.6×10^5 kip
	vertical	=	1.1×10^5 kip	5.5×10^4 kip
	overturning	=	14.4×10^6 kip-ft	25.3×10^4 kip-ft
Seismic	- horizontal	=	2.0×10^4 kip	3.6×10^4 kip
	overturning	=	1.7×10^6 kip-ft	4.9×10^6 kip-ft
Ice	- horizontal	=	2.8×10^4 kip	2.8×10^4 kip
	overturning	=	8.4×10^6 kip-ft	16.8×10^6 kip-ft

See discussion in Section 5.3.2.2.

C. Loading Combinations

	Load					
	Horizontal (kip)		Vertical (kip)		Overturning (kip-ft)	
Case	300 ft.	600 ft.	300 ft.	600 ft.	300 ft.	600 ft.
I	$1.1x10^5$	$1.6x10^5$	$1.4x10^5$	$2.2x10^5$	$14.4x10^6$	$25.3x10^6$
II	----	----	$3.3x10^4$	$1.6x10^5$	----	----
III	$1.1x10^5$	$1.6x10^5$	$2.0x10^5$	$2.5x10^5$	$14.4x10^6$	$25.3x10^6$
IV	$5.5x10^4$	$3.0x10^4$	$2.5x10^5$	$2.8x10^5$	$7.2x10^6$	$12.6x10^6$
V	$2.0x10^4$	$3.6x10^4$	$1.4x10^5$	$2.2x10^5$	$1.7x10^6$	$4.9x10^6$
VI	$2.8x10^4$	$2.8x10^4$	$1.4x10^5$	$2.2x10^5$	$8.4x10^6$	$16.8x10^6$

5.4.2.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structures shown in Figures 5-2 and 5-3.

	Coefficient					
	Sliding		Overturning		Bearing	
Criteria	300 ft. 1.5	600 ft. 1.5	300 ft. 2.0	600 ft. 2.0	300 ft. 2.0	600 ft. 2.0
I	2.2	1.5	2.7	2.4	2.3	1.4
II	---	---	---	---	45.6	9.2
III	2.2	1.5	3.8	2.7	2.9	1.3
IV	4.4	2.9	9.7	6.0	4.4	3.3
V	12.5	6.5	22.5	12.5	9.4	5.6
VI	8.5	8.5	4.6	3.6	7.0	3.9

From the above results, it is observed that the shallower structure (W.D. = 300 ft) is stable in all modes. In the case of the deeper structure (W.D. = 600 ft), the minimum bearing requirement are not satisfied. The bearing is controlled by the maximum wave loads. The results obtained indicate a limiting condition for the selected structural configuration given the specific water depth, environmental loads, and soil properties. It is interesting to note that the reduced seismic exposure in the Navarin Basin as compared to the St. George Basin resulted in satisfactory stability against sliding for the deeper structure. The increased ice exposure in the Navarin Basin required the consideration of ice loading which wa not critical in the St. George Basin. Nevertheless, the waves rather than the ice loading governs the design criteria in the Navarin Basin. Other alternatives such as the tripod gravity base structure (Figure 5-4) may be more suitable for the given design conditions. It should be noted here that the selected structural alternative will fully satisfy the prescribed geotechnical stability criteria in the case of the upper bound clay soils (i.e., undrained shear strength equals 2 ksf).

5.4.2.4 Material Take Off

Total Concrete in 300' Structure	204000 CY
Total Concrete in 600' Structure	325657 CY

Concrete volumes are equal to those found in Section 5.3.2.4 - St. George Basin; Water Depth Variation.

5.4.3 Environmental Conditions Variation

In this case only the ice loads on the base case structure (Figure) are varied. The lower limit specified by the sensitivity range (60 kip/ft) is not considered because it does not constitute a critical loading condition. See Section 5.3.3 for further discussion.

5.4.3.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip
Storage	=	1×10^6 bbl

B. Environmental

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.01g
- velocity	=	2 in/sec
Ice - rafted ice	=	170 kip/ft

C. Geotechnical

Clay - undrained shear strength	=	1.0 ksf

.4.3.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	957000 kip
Minimum ballast required	=	473000 kip
Total air weight	=	1430000 kip

B. Environmental

Oceanographic	- horizontal	=	9.2×10^4 kip
	vertical	=	7.9×10^4 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	2.0×10^4 kip
	overturning	=	1.4×10^6 kip-ft
Ice	- horizontal	=	4.3×10^4 kip
	overturning	=	19.5×10^6 kip-ft

The changes in horizontal and overturning ice loads as compared to the base case are directly proportional to the increase in the ice strength.

C. Loading Summary

Case	Load		
	Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	9.2×10^4	1.1×10^5	8.7×10^6
II	----	5.1×10^4	----
III	9.2×10^4	1.5×10^5	8.7×10^6
IV	4.6×10^4	1.9×10^5	4.4×10^6
V	2.0×10^4	1.1×10^5	1.4×10^6
VI	4.3×10^4	1.1×10^5	19.5×10^6

5.4.3.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-1.

	Coefficient		
Case Criteria	Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.6	3.5	5.7
II	---	---	31.8
III	2.6	4.8	5.2
IV	5.2	12.0	5.3
V	11.9	21.6	12.0
VI	5.6	1.6	2.2

Based on the above results, it is concluded that the proposed structure does not satisfy minimum overturning stability requirements under ice loading. Overturning and bearing are both controlled by the maximum ice loading. The maximum ice loads were conservatively calculated assuming full bridging and arching. This assumption must be further investigated before final assessment is made.

An alternative configuration involving more closely-spaced shafts or a monotower is believed to be capable of fulfilling the design criteria for this case.

5.4.3.4 Material Take Off

Same as base case.

5.4.4 Operating Conditions Variation

Reducing the crude oil production rate from 300000 BPD to 100000 BPD, thereby reducing the payload, has only a marginal effect on the global weight and geotechnical stability of concrete gravity base structures. Hence, this variation was not considered as a separate design case. Nevertheless, changes in the payload capacity may have a significant effect during the transportation and installation phases. Both the motion response during tow and the hydrostatic stability strongly depend upon the magnitude and elevation of the topside payload.

5.4.5 Soil Conditions Variation

In this section the geotechnical stability of the base case structure (Figure 5-1) is evaluated when founded on granular soils (sand). For a complete discussion concerning this variation, see Section 5.3.5.

5.4.5.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. <u>Operating</u>

Payload	=	100,000 kip
Storage	=	1×10^6 bbl

B. <u>Environmental</u>

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.05g(2)
- velocity	=	2 in/sec
Ice - rafted ice	=	110 kip/ft

C. <u>Geotechnical</u>

Sand - internal friction angle	=	30°(1)

.4.5.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	957000 kip
Minimum ballast required	=	803000 kip
Total air weight	=	1760000 kip

B. Environmental

Oceanographic	- horizontal	=	9.2×10^4 kip
	vertical	=	7.9×10^4 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	2.4×10^4 kip
	overturning	=	2.1×10^6 kip-ft
Ice	- horizontal	=	2.8×10^4 kip
	overturning	=	12.6×10^6 kip-ft

The increased ballast and changed soil-structure interaction affect only the seismic loads which are somewhat higher than in the base case.

(1) The internal friction angle was conservatively chosen to represent sands between lower bound and design basis at the skirt tips, 15 feet below the base line.

(2) See note (2) in Section 5.4.1.1.

C. Loading Combinations

Case	Load		
	Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	9.2×10^4	4.4×10^5	8.7×10^6
II	-----	3.4×10^4	-----
III	9.2×10^4	4.8×10^5	8.7×10^6
IV	4.6×10^4	5.2×10^5	4.4×10^6
V	2.6×10^4	4.4×10^5	2.3×10^6
VI	2.8×10^4	4.4×10^5	12.6×10^6

5.4.5.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-1.

Case Criteria	Coefficient		
	Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.8	14.0	32.8
II	---	---	48.5
III	3.0	15.2	30.2
IV	6.5	33.3	29.6
V	10.0	57.7	33.0
VI	9.1	9.6	30.2

Based on the above results, it is concluded that the structure is stable in all modes.

5.4.5.4 Material Take Off

Same as base case.

5.4.6 Storage Conditions Variation

In this section the storage requirements of the base case structure (Figure 5-1) are varied. The base case structure was modified to store 3 million barrels of crude oil (see Figure 5-5). This was accomplished by increasing the base height to 200 feet with the same base plan dimensions. Storage below 1 million barrels would result in savings in the mechanical equipment, but would not reduce the base dimension or concrete volume beyond the base case due to minimum ballast and structural integrity requirements.

5.4.6.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. <u>Operating</u>

Payload	=	100,000 kip
Storage	=	3×10^6 bbl

B. <u>Environmental</u>

Oceanographic - wave height	=	85 ft.
wave period	=	15 sec.
Seismic - peak ground acceleration	=	0.05g(1)
- velocity	=	2 in/sec
Ice - rafted ice	=	110 kip/ft

C. <u>Geotechnical</u>

Clay - undrained shear strength = 1.0 ksf

5.4.6.2 Results of Analysis

A. <u>Operating</u>

Air weight of GBS plus topside	=	1307000 kip
Minimum ballast required	=	1003000 kip
Total air weight	=	2310000 kip

B. <u>Environmental</u>

Oceanographic	- horizontal	=	1.5×10^5 kip
	vertical	=	1.0×10^5 kip
	overturning	=	7.2×10^6 kip-ft
Seismic	- horizontal	=	3.3×10^4 kip
	overturning	=	4.3×10^6 kip-ft
Ice	- horizontal	=	2.8×10^4 kip
	overturning	=	12.6×10^6 kip-ft

(1) See note (2) in Section 5.3.1.1.

C. Loading Summary

Case	Load: Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	1.5×10^4	2.2×10^5	7.2×10^6
II	-----	1.2×10^5	-----
III	1.5×10^4	2.7×10^5	7.2×10^6
IV	0.8×10^4	3.2×10^5	3.6×10^6
V	3.3×10^4	2.2×10^5	4.3×10^6
VI	2.8×10^4	2.2×10^5	12.6×10^6

5.4.6.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-5.

Case Criteria	Coefficient: Sliding 1.5	Overturning 2.0	Bearing 2.0
I	1.6	8.4	2.4
II	---	---	11.9
III	1.6	10.3	2.0
IV	3.1	24.2	3.4
V	7.2	14.1	5.7
VI	8.5	4.8	4.6

From the above results it is observed that the structure is stable in all modes. It is further noted that sliding and bearing are controlled by the maximum wave loading, whereas overturning is controlled by the maximum ice loading.

5.4.6.4 Material Take Off

Total Concrete Volume	310717 CY

Concrete volume is equal to that found in Section 5.3.6.4 - St. George Basin; Storage Conditions Variation.

5.5 North Aleutian Shelf

5.5.1 Base Case

The base case structure for the North Aleutian Shelf is to be founded on granular (sand) soil in 300 feet of water. The structure is designed to accommodate a production rate of 300,000 BPD and provide a storage capacity of 1 million barrels. A general layout of the proposed structural configuration is shown in Figure 5-2. The analytical results given below are primarily to evaluate suitability of the base case design to resist global loads under the specified environmental, operational and geotechnical criteria.

5.5.1.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip(1)
Storage	=	1×10^6 bbl

B. Environmental

Oceanographic - wave height	=	75 ft.(2)
wave period	=	14 sec.

(1) 91,800 kip rounded up.

(2) A wave height of 75 feet was used as opposed to the 78 feet specified as the design basis. This represents only a minor deviation which is insignificant in view of the ample stability of the base case.

Seismic - peak ground acceleration = 0.24g[3]

- velocity = 8 in/sec

Ice - rafted ice = 85 kip/ft

C. Geotechnical

Sand - internal friction angle = 30°[4]

5.5.1.2 Results of Analysis

In conducting the analysis presented here no attempt was made to optimize the structural geometry or weight characteristics. The initial configuration was assumed based on the previous analyses and past experience. The minimum ballast requirements was determined such that sufficient stability against uplift and sliding was obtained. The other possible modes of failure were then analyzed to ensure compliance with the stability criteria.

A. Operating

Air weight of GBS plus topside	=	581000 kip
Minimum ballast required	=	783000 kip
Total air weight	=	1364000 kip

(3) The specified acceleration is the peak ground acceleration; however, the seismic forces were calculated based on the design basis response spectrum. Based on an anticipated fundamental response period of 2 sec. The estimated spectral acceleration from the criterion response spectrum is approximately 0.05 g, but was rounded up to .08 g to account for possible underconservatism in the static analysis method which is based only on the fundamental mode of vibration.

(4) See note (1) in section 5.3.5.2.

B. Environmental

Oceanographic	- horizontal	=	1.1×10^5 kip
	vertical	=	1.1×10^5 kip
	overturning	=	3.6×10^6 kip-ft
Seismic	- horizontal	=	1.5×10^5 kip*
	overturning	=	11.9×10^6 kip-ft*
Ice	- horizontal	=	2.0×10^4 kip
	overturning	=	6.5×10^6 kip-ft

*(Seismic responses may be low for denser sands.)

C. Loading Summary

Case	Load Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	1.1×10^5	3.7×10^5	3.6×10^6
II	-----	2.6×10^5	-----
III	1.1×10^5	4.3×10^5	3.6×10^6
IV	5.5×10^4	4.8×10^5	1.8×10^6
V	1.5×10^5	3.7×10^5	11.9×10^6
VI	2.0×10^4	3.7×10^5	6.5×10^6

5.5.1.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-2.

Case Criteria	Sliding 1.5	Overturning 2.0	Bearing 2.0
	Coefficient		
I	2.0	25.2	28.1
II	---	---	42.4
III	2.3	28.9	24.4
IV	5.1	65.1	22.8
V	1.5	7.6	23.1
VI	10.0	14.0	26.2

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding and overturning are controlled by the maximum seismic loading. Bearing is controlled by the wave loading.

A sand with internal angle of friction of only 30° is possibly subject to liquefaction under earthquake. Stability under strong seismic action must be evaluated. Liquefaction potential can be reduced by dynamic consolidation of the sands, blanketing with rock (gravel or crushed rock), or drainage to reduce pore pressures.

.5.1.4 Material Take Off

North Aleutian Shelf - Base Case

1.	Base slab	32835 CY
	Enlargement at base-skirt connection	1000 CY
2.	Interior cell walls	41056 CY
	Enlargement at base-wall connection	1000 CY
3.	Exterior cell walls	8061 CY
	Exterior cell walls	9069 CY
4.	Roof of cells	9407 CY
5.	Towers	17760 CY
6.	Apron	7000 CY
7.	Ring beam	1000 CY
8.	Miscellaneous	10000 CY
		138188 CY

5.5.2 Water Depth Variation

The water depth upper and lower limits in the North Aleutian shelf are 350 feet and 200 feet, respectively. These cases were not analyzed. However, based on the St. George and the Navarin Basin results as well as the North Aleutian shelf base case design, it is concluded that gravity structures can be installed in any water depth between the above limits. This conclusion is particularly valid due to the fact that the foundation soil in the North Aleutian Shelf is granular (sand) which normally exhibits higher load resistance capacities then undrained cohesive (clay) soils.

5.5.3 Environmental Conditions Variation

In this case only the ice load on the base case structure (Figure 5-2) is varied. This is done by increasing the ice strength from 85 k/ft to 110 k/ft. The lower limit specified by the sensitivity range (55 kip/ft) is not considered because it does not constitute a critical loading condition. The seismic spectral acceleration was not increased because, as explained in Section 5.5.1.1, the initial assumption was already thought to be conservative.

5.5.3.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. Operating

Payload	=	100,000 kip
Storage	=	1×10^6 bbl

B. Environmental

Oceanographic - wave height	=	75 ft.
wave period	=	14 sec.
Seismic - peak ground acceleration	=	0.024g(1)
- velocity	=	8 in/sec
Ice - rafted ice	=	110 kip/ft

(1) See note (3) in Section 5.5.1.1.

C. Geotechnical

Sand - internal friction angle ϕ = 30°

5.5.3.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	581000 kip
Minimum ballast required	=	783000 kip
Total air weight	=	1364000 kip

B. Environmental

Oceanographic	- horizontal	=	1.1×10^5 kip
	vertical	=	1.1×10^5 kip
	overturning	=	3.6×10^6 kip-ft
Seismic	- horizontal	=	1.5×10^5 kip
	overturning	=	11.9×10^6 kip-ft
Ice	- horizontal	=	2.8×10^4 kip
	overturning	=	8.4×10^6 kip-ft

The changes in the horizontal and overturning ice loads as compared to the base case are directly proportional to the increase in the ice strength.

C. Loading Summary

Case	Load: Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	1.1×10^5	3.7×10^5	3.6×10^6
II	-----	2.6×10^5	-----
III	1.1×10^5	4.3×10^5	3.6×10^6
IV	5.5×10^4	4.8×10^5	1.8×10^6
V	1.5×10^5	3.7×10^5	11.9×10^6
VI	2.8×10^4	3.7×10^5	8.4×10^6

5.5.3.3 Preliminary Design

The following safety coefficients were obtained for the structure shown in Figure 5-2.

Case Criteria	Coefficient: Sliding 1.5	Overturning 2.0	Bearing 2.0
I	2.0	25.2	28.1
II	---	---	42.4
III	2.3	28.9	24.4
IV	5.1	65.1	22.8
V	1.5	7.6	23.1
VI	6.6	10.8	25.0

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding and overturning are controlled by the maximum seismic loading. Bearing is controlled by the wave loading.

5.5.3.4 Material Take Off

Same as base case.

5.5.4 Operating Conditions Variation

Reducing the crude oil production rate from 300000 BPD to 100000 BPD and thereby reducing the payload has only a marginal effect on the global weight and geotechnical stability of concrete gravity base structures. Hence, this variation was not considered as a separate design case. Nevertheless, changes in the payload capacity may have a significant effect during the transportation and installation phases. Both the motion response during tow and the hydrostatic stability strongly depend upon the magnitude and elevation of the topside payload.

5.5.5 Soil Condition Variation

No variation in the soil conditions is considered. This is because only soil type B (granular) was identified in the design criteria for the North Aleutian Shelf. Furthermore, the assumed internal friction angle (30°) represents soils close to the lower bound of the design basis with ample stability. However, a higher angle of internal friction would give greater capacities in bearing and sliding resistance, as well as reducing the potential for liquefaction.

5.5.6 Storage Conditions Variation

In this section the storage requirements of the base case structure (Figure 5-2) are varied. The base case structure was modified to store 2 million barrels of crude oil (see Figure 5-6). This was accomplished by increasing the base height to 200 feet with the same base plan dimensions. To accommodate 3 million barrels storage, the base plan dimensions need to be enlarged, resulting in larger environmental forces as well as additional ballast. This larger structural configuration was not investigated. Storage below 1 million barrels would result in savings in the mechanical equipment, but would not reduce the base dimension or concrete volume beyond the base case due to minimum ballast and structural integrity requirements. For detailed calculations for this case, see Appendix.

5.5.6.1 Design Conditions

The specified design conditions include the following operating and environmental requirements as well as the given geotechnical characteristics:

A. <u>Operating</u>

Payload	=	100,000 kip
Storage	=	2×10^6 bbl

B. Environmental

Oceanographic - wave height	=	75 ft.
wave period	=	14 sec.
Seismic - peak ground accelerations	=	0.24g(1)
- velocity	=	8 in/sec
Ice - rafted ice	=	85 kip/ft

C. Geotechnical

Sand - internal friction angle = 30°

5.5.6.2 Results of Analysis

A. Operating

Air weight of GBS plus topside	=	798000 kip
Minimum ballast required	=	1292000 kip
Total air weight	=	2090000 kip

B. Environmental

Oceanographic	- horizontal	=	1.8×10^5 kip
	vertical	=	2.2×10^5 kip
	overturning	=	8.7×10^6 kip-ft
Seismic	- horizontal	=	2.2×10^5 kip
	overturning	=	27.4×10^6 kip-ft
Ice	- horizontal	=	2.0×10^4 kip
	overturning	=	6.5×10^6 kip-ft

(1) See note (3) in Section 5.5.1.1

The increase in the oceanographic and seismic loads as compared to the base case is primarily due to the larger size of the base caisson.

C. Loading Summary

Case	Load Horizontal (kip)	Vertical (kip)	Overturning (kip-ft)
I	1.8×10^5	5.8×10^5	8.7×10^6
II	-----	3.6×10^5	-----
III	1.8×10^5	6.9×10^5	8.7×10^6
IV	0.9×10^5	8.0×10^5	4.4×10^6
V	2.2×10^5	5.8×10^5	27.4×10^6
VI	2.0×10^4	5.8×10^5	6.5×10^6

5.5.6.3 Preliminary Design Evaluation

The following safety coefficients were obtained for the structure shown in Figure 5-6.

Case Criteria	Coefficient Sliding 1.5	Overturning 2.0	Bearing 2.0
I	1.9	16.2	17.2
II	---	---	31.4
III	2.3	19.2	14.8
IV	5.3	44.6	5.9
V	1.5	5.1	12.8
VI	15.5	21.5	17.9

Based on the above results, it is concluded that the proposed structure is stable in all modes. It is also observed that sliding and overturning are controlled by the maximum seismic loading. Bearing is controlled by the wave loading.

5.5.6.4 Material Take Off

North Aleutian Shelf - Storage Condition Variation

1.	Base slab	32835 CY
	Enlargement at base-skirt connection	1000 CY
2.	Interior cell walls	821128 CY
	Enlargement at base-wall connection	1000 CY
3.	Exterior cell walls	16122 CY
	Exterior cell walls	18138 CY
4.	Roof of cells	9407 CY
5.	Towers	11058 CY
6.	Apron	7000 CY
7.	Ring beam	1000 CY
8.	Miscellaneous	10000 CY
		189672 CY

5.6 Summary of Critical Safety Coefficients

The critical safety coefficients for the various cases analyzed in the three basins are summarized and shown in Figures 5-7, 5-8 and 5-9. The assumed stability criteria is indicated by the heavy line. Hence, any coefficient below this heavy line indicates an unstable condition. A review of Figures 5-7, 5-8 and 5-9 indicates the following:

1. Two unstable conditions occur in the St. George Basin. The first condition is in the case of deep water (600 feet) and the second is in the case of the large storage requirement ($3x10^6$ barrels). The deep water structure is unstable due to excessive sliding and bearing which indicates that the selected structural configuration may not be optional for deep water under the prescribed environmental, operating and soil criteria. The large storage requirements result in excessive bearing under the seismic load. Here again, an alternative configuration such as a tripod or quadripod may prove more suitable for these extreme conditions.

2. Two unstable conditions occur in the Navarin Basin. The first condition is in the case of deep water (600 feet). Unlike the corresponding case in the St. George Basin, the bearing requirement is exceeded due to the maximum wave loading. Hence, the applicability of the selected configuration to deep water under the prescribed criteria is in question. The second unstable conditions occurs due to the upper bound ice loading on the structure. This results in an instability due to excessive overturning. The ice loads in this case were conservatively

evaluated assuming full bridging and arching (Section 5.4.3.3). Reduced loading will, of course, result in increased safety. Alternatively the base area can be enlarged to mobilize auxiliary resistance. As noted earlier, a monotower may prove more suitable to the criteria.

3. All the cases evaluated in the North Aleutian Basin indicated ample stability. This is primarily due to the granular soils which enable mobilizing additional stability by means of increased ballast. The design approach was then to provide sufficient ballast such that the controlling sliding requirements are met. Overturning and bearing do not present any difficulties.

   A special note has been added concerning the potential for liquefaction.

5.7 Structural Details

Several typical structural details and sections are shown in Figure 5-10 through 5-14. Figure 5-10 indicates a potential arrangement for the skirts underneath the base. Figures 5-11 and 5-12 show typical sections at exterior and interior base cells. Both sections also idicate the wall thicknesses and typical reinforcement layouts assumed in the foregoing analyses. Partial plan views and sections at the roofs of the exterior and interior base cells are shown in Figure 5-13. Figure 5-14 shows typical sections at the shafts with possible reinforcement details at the ice contact zone.

5.8 Construction Planning

One of the key features of the concrete gravity base structure is that the structure will be completely fabricated and the deck and most of the equipment completely installed and hooked-up in a warm water port, prior to final tow to the site for installation. On-site construction activities are thus reduced to a minimum.

5.8.1 Sequence of Construction

The sequence of construction is schematically illustrated by Drawing 5-15. This series of stages and events is very similar to that which has been carried out on 11 of the large concrete offshore platforms in the North Sea, including those in the Brent, Beryl, Ninian and Statfjord fields. The latest platform, Statfjord C, now under construction in Stavanger Norway is following a similar sequence.

Briefly described in words, the sequence is as follows:

a. A construction basin or existing graving dock is readied for construction.

b. The base raft, including any protruding skirts, is fabricated in the basin.

c. The basin is flooded, the closure dike or gates removed, and the base raft towed to a deep water protected site nearby, where it is moored.

d. The remainder of the base caisson is built up, using the base raft to provide support. As the caisson grows upward, the base sinks deeper in the water.

e. Then the tower shafts are constructed upward from the base caisson.

f. During and after these phases, the mechanical piping and electrical and instrumentation systems within the caisson and shafts are installed.

g. Meanwhile the deck structure is being fabricated at a shipyard, usually temporarily supported on short concrete pillars in shallow water. The pillars duplicate the upper portion of the shafts constructed in (e).

h. Prefabricated modules are placed on the deck supports and integrated into the deck. Additional process and drilling equipment is installed.

i. Hook-up of the deck facilities is carried out at the shipyard.

j. The deck structure, virtually complete is transferred to a large barge or barges. (These may be tanker mid-bodies or similar.)

k. Now the concrete base, complete, is towed to a protected site having very deep water. The deck, complete, is towed to the same site.

l. The concrete base is ballasted down until almost awash. The deck is brought in over the top and carefully positioned.

m. The base is now deballasted, lifting the deck off the transport vessel(s).

n. The deck is post-tensioned to the base, and hook-up completed.

o. The entire structure is now towed to the installation site.

p. At the site, the platform is ballasted down to touch the seafloor. Additional controlled ballasting enables leveling and causes the skirts to penetrate.

q. A weak grout is now pumped in under the base so as to give uniform bearing.

r. Conductors are spudded in through penetrations in the base slab and drilling commences.

s. As wells are drilled, production commences. After all wells are drilled, part of the drilling equipment may be removed.

t. When the field has been commercially exhausted (e.g., in 20 to 30 years), the drilling and process equipment can be removed. The structure is then deballasted. Water is flooded in under the base and the structure raised off the seafloor for tow to disposal or reuse site.

A variation available for items b and c, where it is necessary or desirable to reduce the construction basin size, is to construct the base raft in several segments. After all segments are launched, they are brought together by use of mating cones and rubber seals, just as has been done on many subaqueous tubes and floating bridges. The joints are now fixed with grout and post-tensioning.

Other acceptable "state-of-art" modifications include the use of an air cushion in Step C, under the base and confined by the skirts, so as to reduce draft during float out.

5.8.2 Criteria for Selection of Sites for Fabrication and Deck Mounting

The construction basin must be large enough to accommodate the base and deep enough to enable sufficient structural depth to be constructed prior to float-out, in order to ensure adequate strength and freeboard to support the construction of the caisson.

Most such basins have been constructed in the open, that is with clay, sand, or rock bottom and sides. Thus extensive sheet, piling cut-off walls or dewatering systems may be required.

Substantial facilities are required at the construction basin site: concrete plant(s), warehouses for reinforcing and prestressing steel, cranes, utilities, offices etc. At remoter locations, a camp may also be required.

Next a deep water construction site must be available, protected from waves sufficiently to enable construction operations to be carried on substantially continuously. This site should preferably be in the vicinity of the construction basin to enable joint use of the support facilities.

Floating concrete plant(s), barges, crew boats, etc. are required.

The deck mating site must be characterized by water depth deep enough to enable the concrete structures to be ballasted down almost awash. It should also be protected from severe exposure to wind and waves during the mating period.

The deck fabrication site should preferably be at or adjacent to a large shipyard or similar industrial facility. If remote from a shipyard, then camp and support facilities are required.

5.8.3 Other Aspects

At the primary construction sites, there must be made available the requisite supplies of concrete aggregate, cement, and steel, as well as an adequate supply of semi-skilled and skilled construction labor. Facilities must be available to receive, store, fabricate, and coat the piping. Construction plant and equipment must also be made available: fortunately, it is rather mobile.

At the deck fabrication and assembly site, skilled welders, riggers, and mechanical and electrical trades must be available. They must have the necessary support in the form of equipment and facilities. Hence, a site in or adjacent to an existing large shipyard is ideal.

5.8.4 Candidate Sites

Several sites have been identified as appropriate to the Southern Bering Sea development. Figure 5-16 presents construction sites and towing routes discussed in the following text.

A. Puget Sound

This is the only location in the lower 48 U. S. states which resembles the Norwegian fjords and Scottish lochs. It is blessed by deep protected waters, excellent concrete aggregate, a strong industrial infrastructure, and an abundance of semi-skilled and skilled labor.

However, there are some physical and environmental restrictions.

At Tacoma, there already exists one construction basin 600 feet square, and another 160 feet wide by 600 feet long. Both would have to be deepened. Downstream from both is an existing bridge with 144 feet ± horizontal clearance.

Hence, the base raft would have to be constructed in segments, to be joined in the Tacoma Outer Harbor.

Anacortes appears to be another highly suitable site, zoned industrial, with deep water close offshore, excellent transport, and a moderate supply of labor.

Other potential sites exist: Bremerton, Everett, Port Townsend.

The State of Washington and the Puget Sound Area in particular are very environmentally conscious and permits even for such apparently nonpolluting activities as construction afloat may require a considerable period of time. Hence, priority should be directed to areas such as the Tacoma Outer Harbor, Anacortes, and Bremerton, where existing similar activities have been carried out over the years.

The Tacoma area is blessed with some of the finest concrete aggregates in the world, and several contractor organizations with extensive experience in sophisticated concrete construction. The ARCO Ardjuna LPG barge, now in the Java Sea, the Hood Canal Floating Bridge, and the Valdez floating container terminal were all constructed here.

To tow from Tacoma or Bremerton to the sea, requires crossing Admiralty Bar, with a limiting water depth of 37 fathoms (about 220 meters).

Deep water deck mating areas exist seaward of Admiralty Bar between Port Townsend and Port Angeles. These sites are relatively protected from extreme storms, but have a long fetch to the Northwest.

The San Juan Islands offer deep water, without restriction for exit to the ocean, but are very sensitive areas for boating.

Exit from the Port Angeles or San Juan Island area is through the Straits of San Juan de Fuca, having over 330 ft depth all the way, with, however, some rather tortuous navigation in the open ocean just outside if it is necessary to stay in water deeper than 330 ft.

Qualified contractors are available with experience in large-scale sophisticated concrete construction. These include:

Concrete Technology Corp.
Morrison-Knudsen
Dillingham Corp.
Peter Kiewit Sons' Co.
Guy F. Atkinson Co.
J. A. Jones Construction Co.
J. H. Pomeroy Co.
Reidel International

For transfer of such heavy lifts as the deck, on the West Coast are the following organizations:

Rig International

Tokola Offshore, Inc.

B. California

Potential construction basin sites exist in the San Francisco Bay Area, e.g., at Rio Vista. From there, the base segments would have to be towed to Long Beach Harbor, where they could be assembled and jointed.

(From time to time similar small basin areas become available on Terminal Island, Los Angeles Harbor, and in San Diego, south of the Coronado Bridge.)

In Long Beach Harbor, with over 50 feet of protected water, the caisson would be constructed to a total height of above 100 ft. From there it could be towed to protected water to the east of Santa Rosa Island, the site used in the assembly of the two halves of the Exxon Hondo platform.

Construction could be completed afloat at this site, followed by deck mating at the same site. The major problem is the 20 miles open water distance from a support base (Santa Barbara or Port Hueneme), which makes crew changes and resupply awkward and costly.

C. Alaska

Previous in-depth studies and actual experience have shown that the cost of construction in Alaska is approximately twice that in the lower 48 states, and that prestressed concrete structures can most economically be constructed in Puget Sound and towed to Alaska.

However, at least one excellent construction site, from the physical point of view, does exist in Resurrection Bay, near Seward, Alaska. The water is deep and generally protected (Figure 5-17).

At Fourth of July Creek in Resurrection Bay, a deltaic deposit of sands and gravels would furnish a fine site for a construction basin, although extensive dewatering would be required.

There is some question as to whether or not the local sands and gravels near Seward are of sufficiently high quality for concrete of the strengths needed. It could conceivably prove necessary to import aggregates from Portage or Anchorage to Seward.

Seward has excellent rail and water connections, is the terminus of the Alaska Railroad, and has many local facilities. A camp would have to be set up for labor.

Seward experiences lower temperatures than Puget Sound, although no sea ice forms. It is an open port all year. Nevertheless, the cold winds in the winter could require protection for workers and concrete (Figure 5-18).

The fjord-like bay is also subject to sudden williwaw winds, with high gusts.

Labor rates in Alaska are roughly 50 percent greater than in Puget Sound.

The effect of all these is to significantly increase the cost of construction in Alaska over that of Puget Sound: a factor of 2.0 to 2.2 is often applied.

D. Japan

From Japan to the Southern Bering Sea is about 500 miles further than from Puget Sound. However, the summer currents are favorable, rather than adverse, so that total tow time should not be much greater.

In Japan, there exist six graving docks, with gate widths of about 330 ft and draft over the sill of 140 ft. These docks were built for the purpose of constructing supertankers and are now converted to building smaller ships. Lead time for availability is currently 18 months.

Mitsui Shipbuilding has a suitable site for a construction basin large enough to accommodate the largest structures proposed in this report. They would probably have the concrete construction carried out by Kajima Construction Co., under subcontract or Joint Venture arrangement.

Deep water mating sites exist near Shimizu, on Honshu Island, between Nagoya and Tokyo, and near Yokosuka, the U. S. naval base at the entrance to Tokyo Bay.

It is believed that other Japanese contractors such as Penta-Ocean are also making plans to be able to carry out such construction activities.

The availability of large shipyard facilities for construction of the deck and assembly of process equipment is of course excellent, and a number of sites at Osaka and west, on the Inland Sea, are available.

E. Korea

Finally, the possibility of construction in South Korea, near Pusan, must be considered, in view of their almost explosive development of large shipbuilding and their experience with such complexes as the Prudhoe Bay Waterflood Facility. Hyundai is a potential contractor with extensive prestressed concrete capability and experience. Dae Woo has shipyards and steel capability.

F. Shipyards

Shipyard facilities suitable for deck construction exist at Todd Shipyards, Los Angeles; National Shipbuilding San Diego; at Lockheed Shipyard, Seattle; and at Tacoma Boat Works, Tacoma. At all these sites, space will be crowded, especially if other activities are going on at the same time.

Excellent space and facilities exist at Hunter's Point, San Francisco (the old San Francisco Naval Shipyard), but the completed deck would then have to be towed to Santa Rosa Island, necessitating a relatively hazardous exist across the San Francisco Bay entrance bar.

5.8.5 Towage

A. U. S. Pacific Coast

From the U. S. Pacific Coast, i.e., Puget Sound or Southern California, the route is essentially west across the North Pacific to either Unimak or Amukta Pass. Unimak is a relatively shallow pass through the Aleutian Islands, with strong tidal currents. For the larger structures for the St. George and Navarin Basins, therefore, it appears that Amukta Pass, further west, is best. For the North Aleutian Basin, with shallower draft structures, it should be practicable to use Unimak Pass.

Current tug availability of U. S. flag boats, required for tow from one U. S. port to a U. S. offshore site by the Jones Act, consists of 9000-10000 H.P. boats. Discusssions with Crowley Maritime, the largest operator, indicate he probably would build larger boats if a major development took place.

In any event, we envision a tow not unlike that for the Ninian Central Platform, with five main towing boats.

Refueling underway would be carried out by an additional boat.

Across the North Pacific, there is a generally opposing current of about one knot in the summer. It could be expected that at least one summer storm will be encountered, with seas having an H_s of 20 ft and a period of about 12 seconds.

Sudden summer storms are often encountered also in the Southern Bering Sea, but periods of relative calm do exist in the summer and early fall, making it practicable to install a gravity platform with relative ease.

B. Japan

Towage from Japan would probably proceed easterly on the 40th parallel to about the Date Line, thence turn north, penetrating the Aleutian chain at Amchitka Pass. This would take advantage of favorable currents and more favorable weather. The distance is farther than from Puget Sound by 500 miles or so but the favorable currents partially offset this.

No restrictions exist on the use of foreign boats when towing from Japan to a U. S. site; hence, the very large Dutch and German tugs of 20000 H.P. can be used. It is probable that, with an order, Japan would build larger (22,000 I.H.P.) tugs for the service. Three to four would probably be used.

C. Alaska

Because the route from Resurrection Bay is southwest, the saving in tow distance is only about 1000 miles as compared with Puget Sound. Favorable currents will be found for the first part of the trip and only about 600 miles are exposed to Gulf of Alaska summer storms.

One scenario that should be kept in mind is to build the concrete structure in Puget Sound, the integrated deck in Japan, and perform the mating in Resurrection Bay.

5.8.6 Installation

On arrival at the site, the boats fan out in star-like formation. Pontoon buoys may be used in the lines to prevent pulling the stern of the boat underwater, due to the steep angle of the line to the top of the caisson.

The structure is ballasted down to touch, then penetrated, and finally grouted under the base (see Steps p, q and r of Section 5.8.1.1.)

5.9 Conclusions

The concrete gravity-based structure, similar to those currently being installed in the North Sea (e.g., Statfjord C), appears well adapted to most of the alternative combinations of criteria present in the Southern Bering Sea. The major advantages are the ability to mount and transport the deck payload of processing and drilling equipment, and the short time required to install a structure ready to resist major environmental forces. Tables 5-2, 5-3 and 5-4 provide a general summary of the base case and variations discussed for each basin.

Under certain combinations of the controlling parameters, the required safety factors were not met. The four limiting parameters were found to be, in order of criticality; very soft soils, storm waves, seismicity, and ice loading. Under such criteria, alternative configurations such as the monotower (to reduce ice loading) or the tripod or quadripod appear to be more suitable. The latter, exemplified by the Condeep T-300, for example, is believed to be suitable for these extreme cases.

Because of the sensitivity of these structures to the seismic parameters at each site, a more detailed investigation should be carried out to determine seismic parameters such as velocity, displacement, and duration. Geotechnical conditions are also very important in determining not only bearing and sliding capacity, but seismic response and damping.

Recognizing the need for site-specific designs as indicated above, the general concept of a concrete gravity-based platform appears to offer significant benefits for offshore operations in the Southern Bering Sea, especially those which are more remote, such as the northern St. George and Navarin Basins.

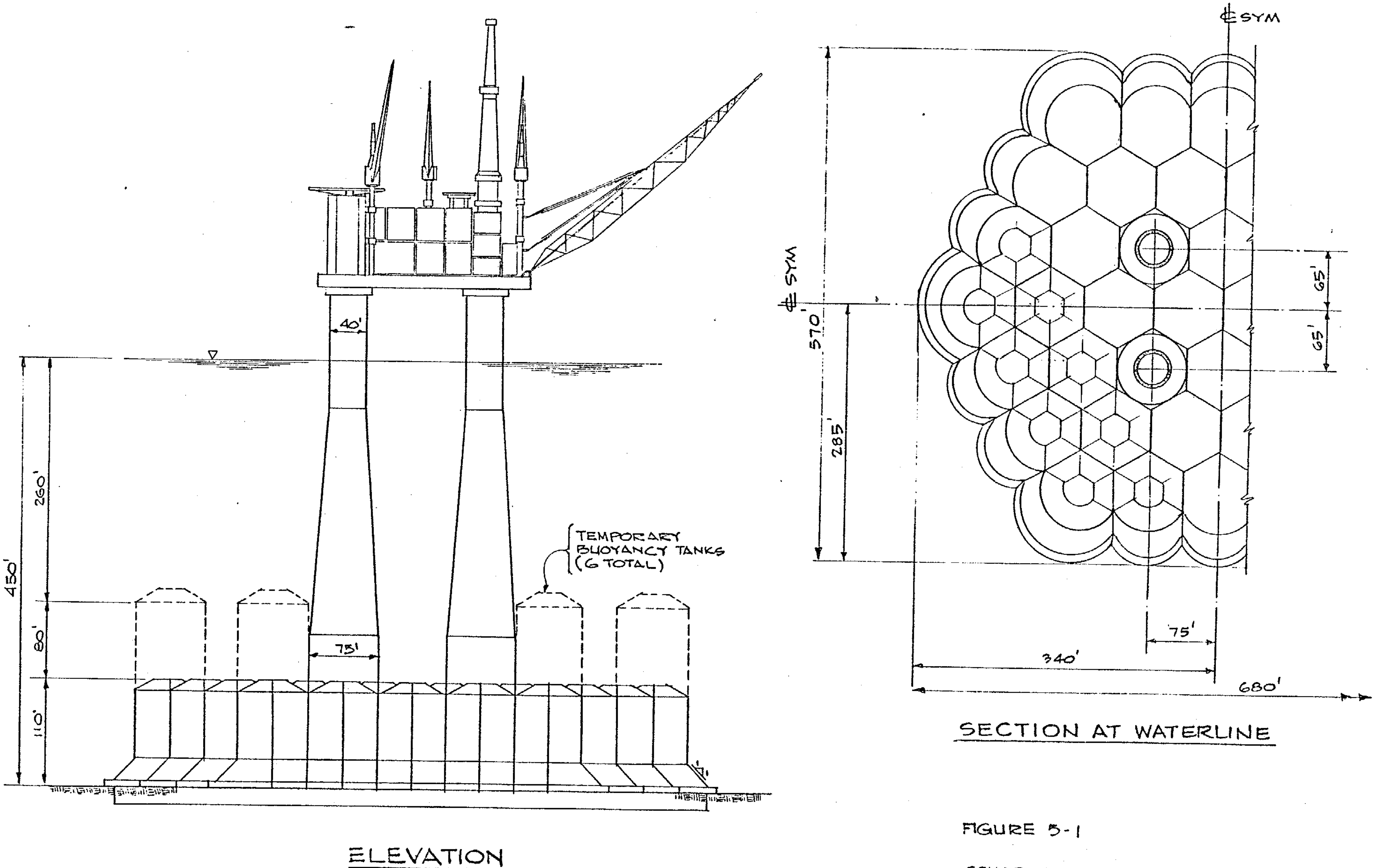
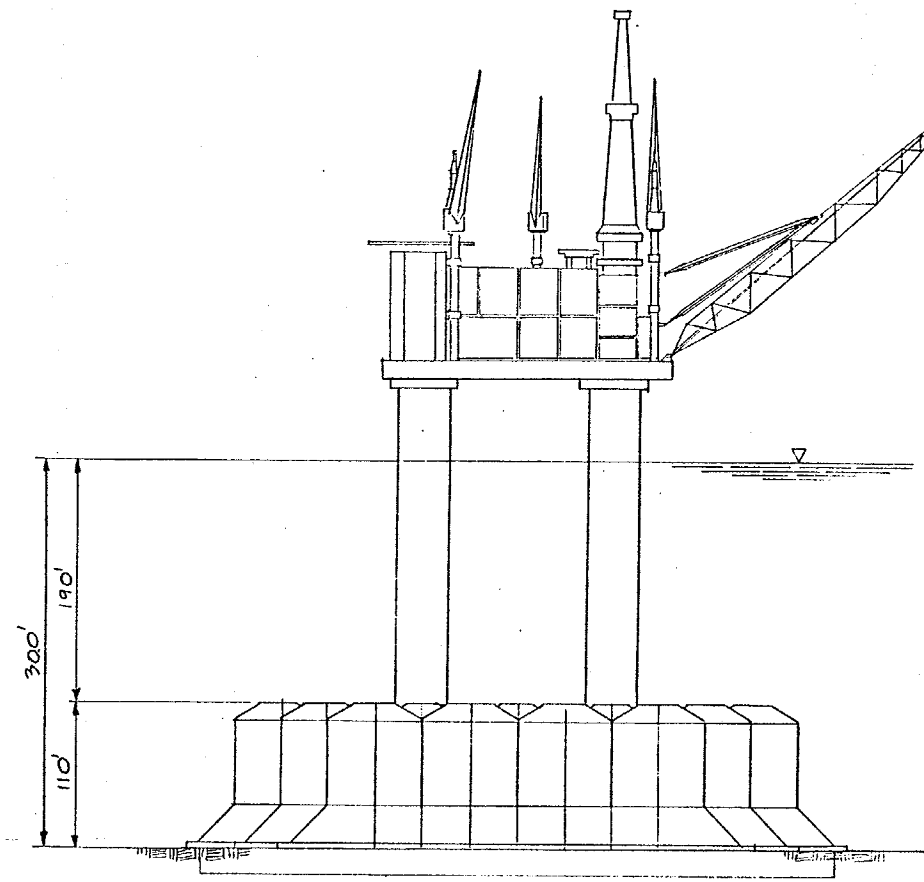
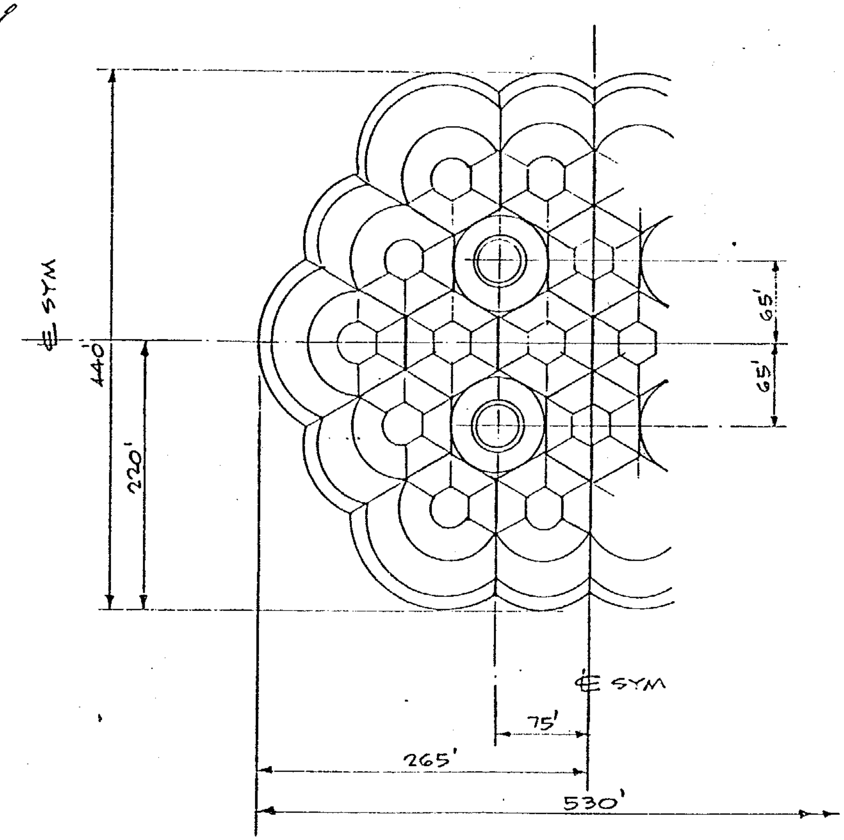


FIGURE 5-1

CONCRETE GBS
WATER DEPTH = 450 FT.
STORAGE CAPACITY = 1,000,000 BBL.



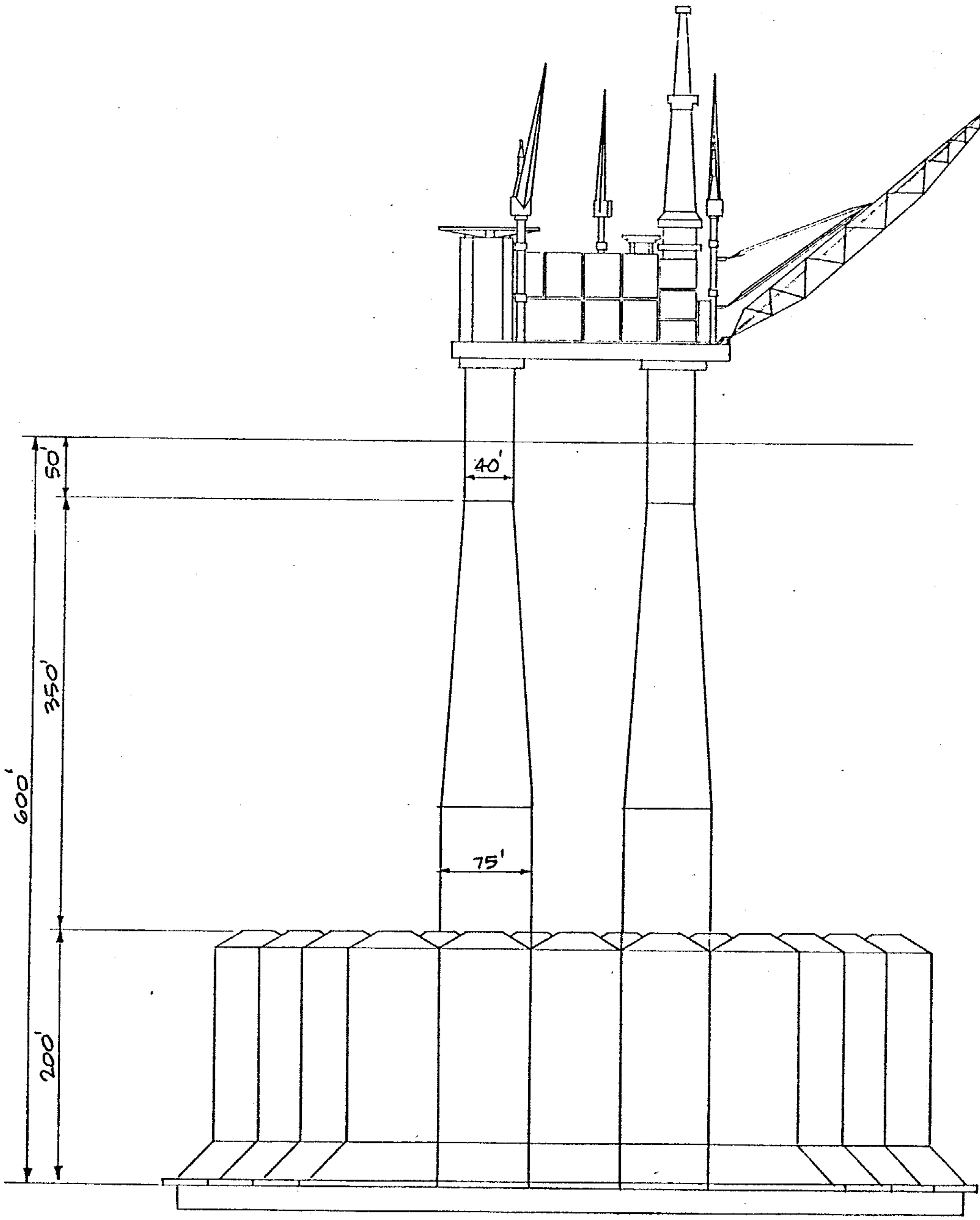
ELEVATION



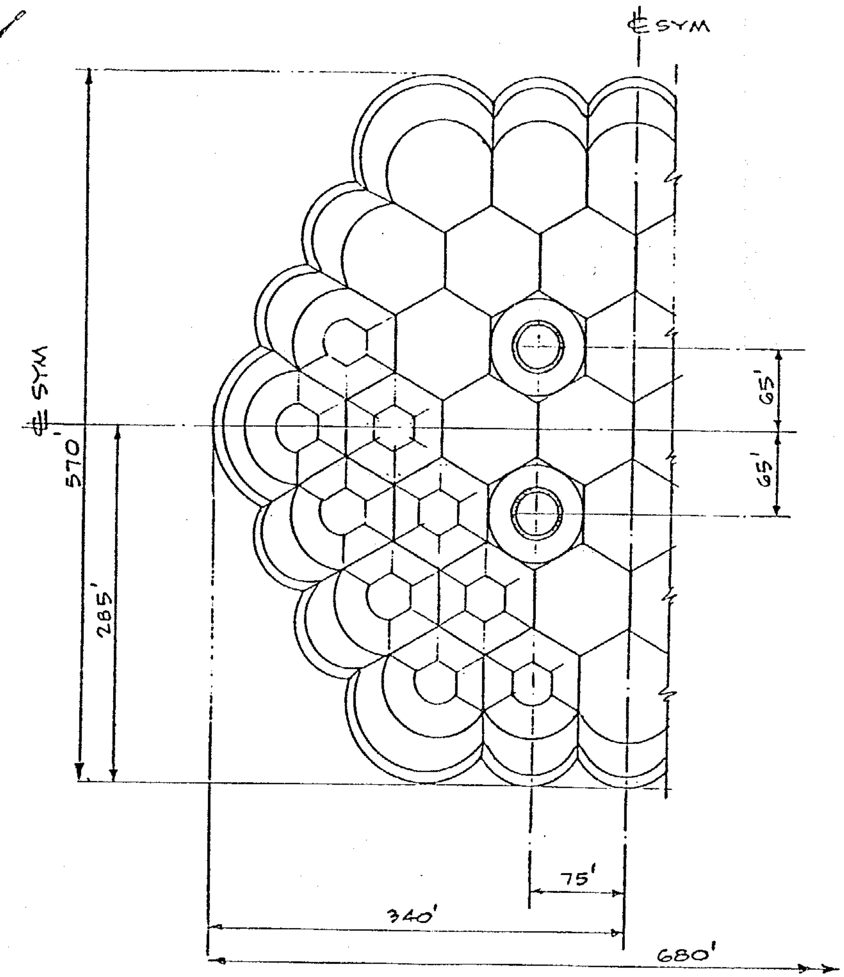
SECTION AT WATERLINE

FIGURE 5-2

CONCRETE GBS
WATER DEPTH = 300 FT.
STORAGE CAPACITY = 1,000,000 BBL



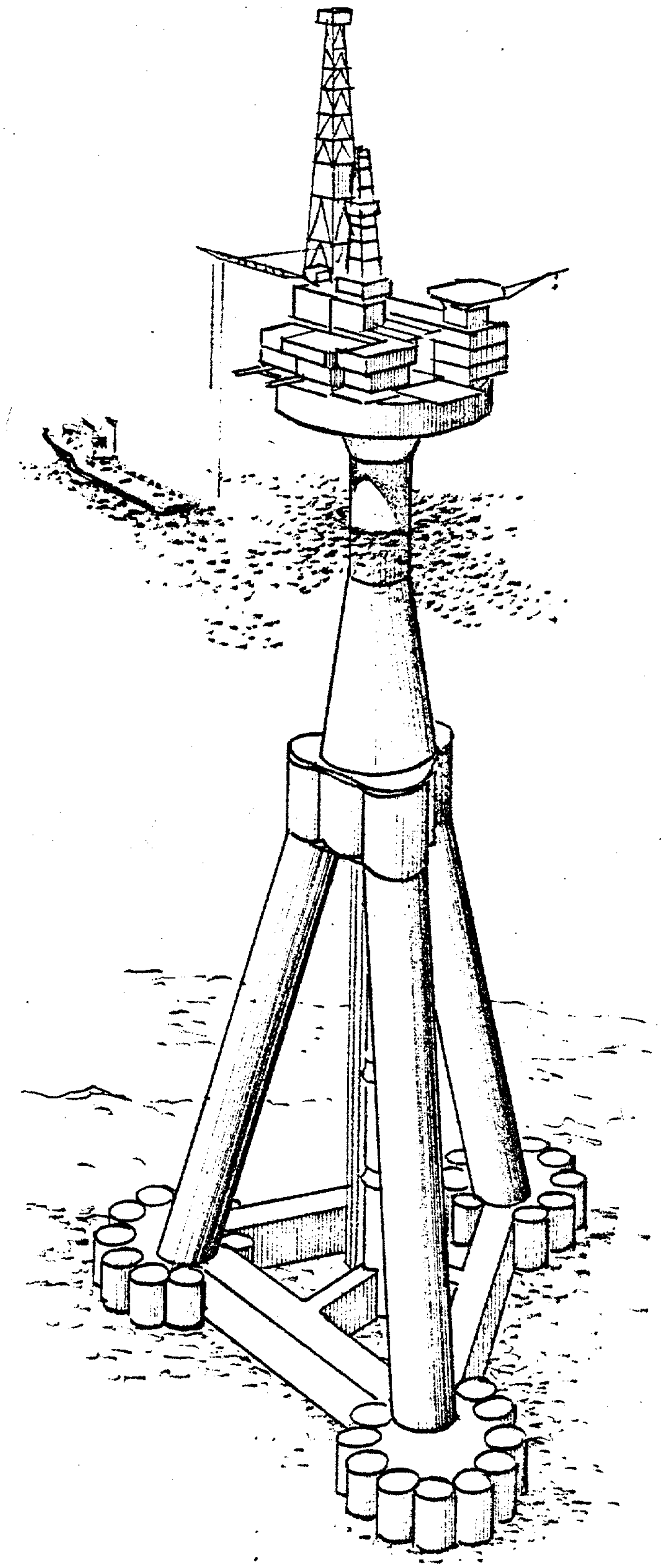
ELEVATION



SECTION AT WATERLINE

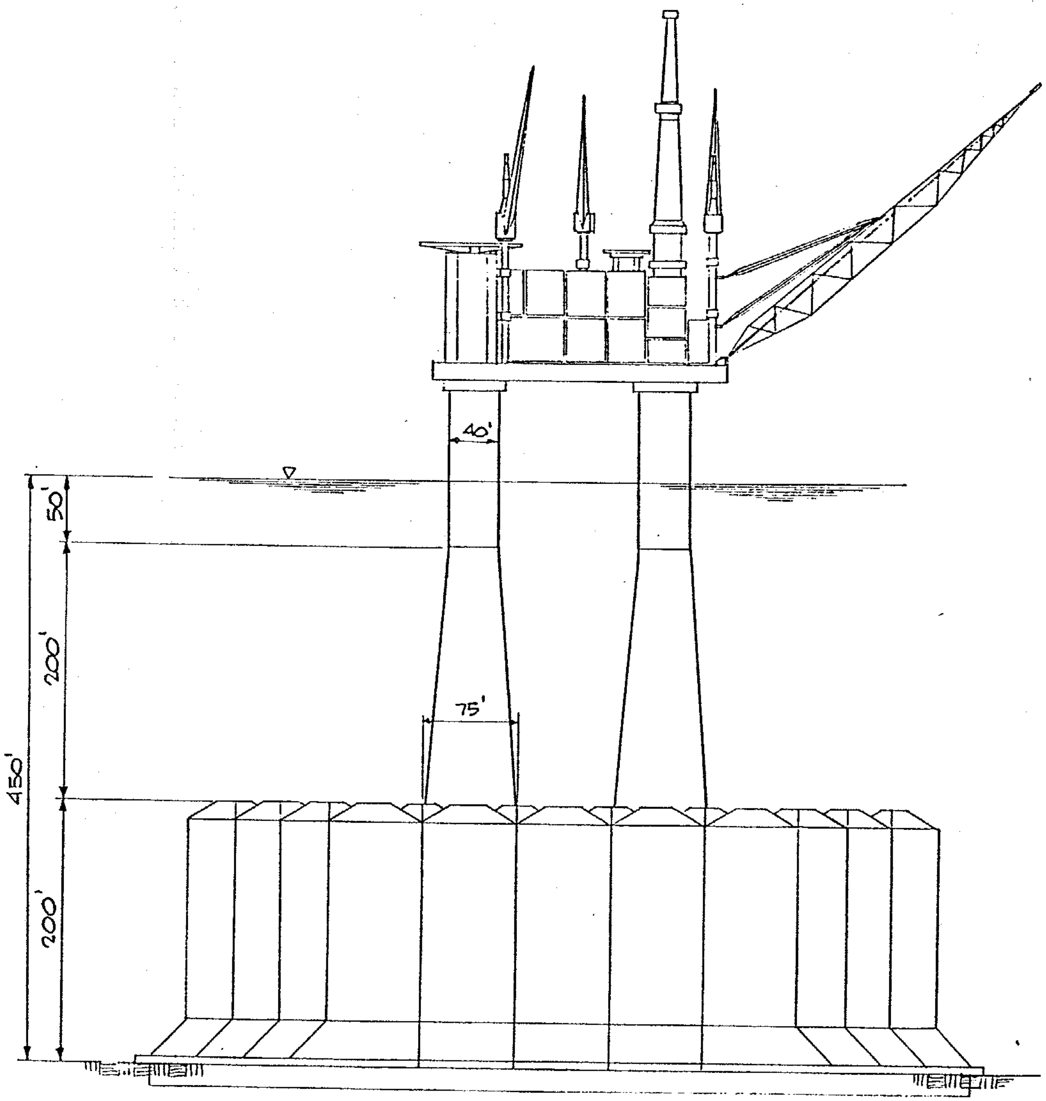
FIGURE 5-3

CONCRETE GBS
WATER DEPTH = 600 FT.
STORAGE CAPACITY = 3,000,000 BBL

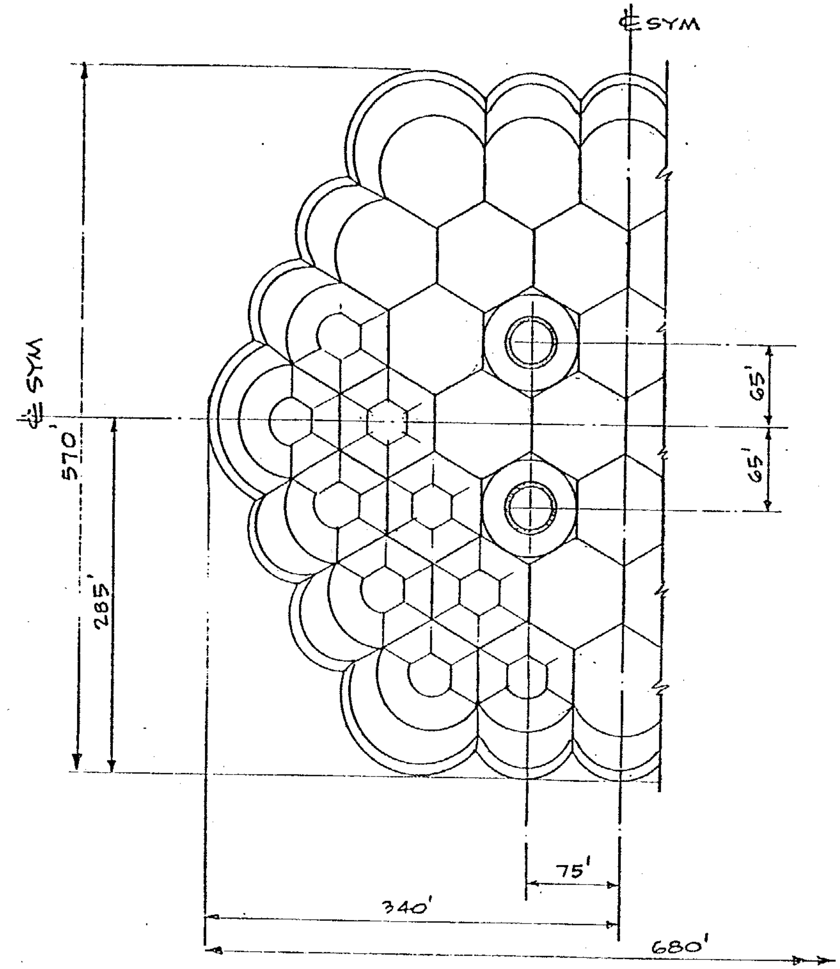


TRIPOD GRAVITY BASE STRUCTURE

FIGURE 5-4



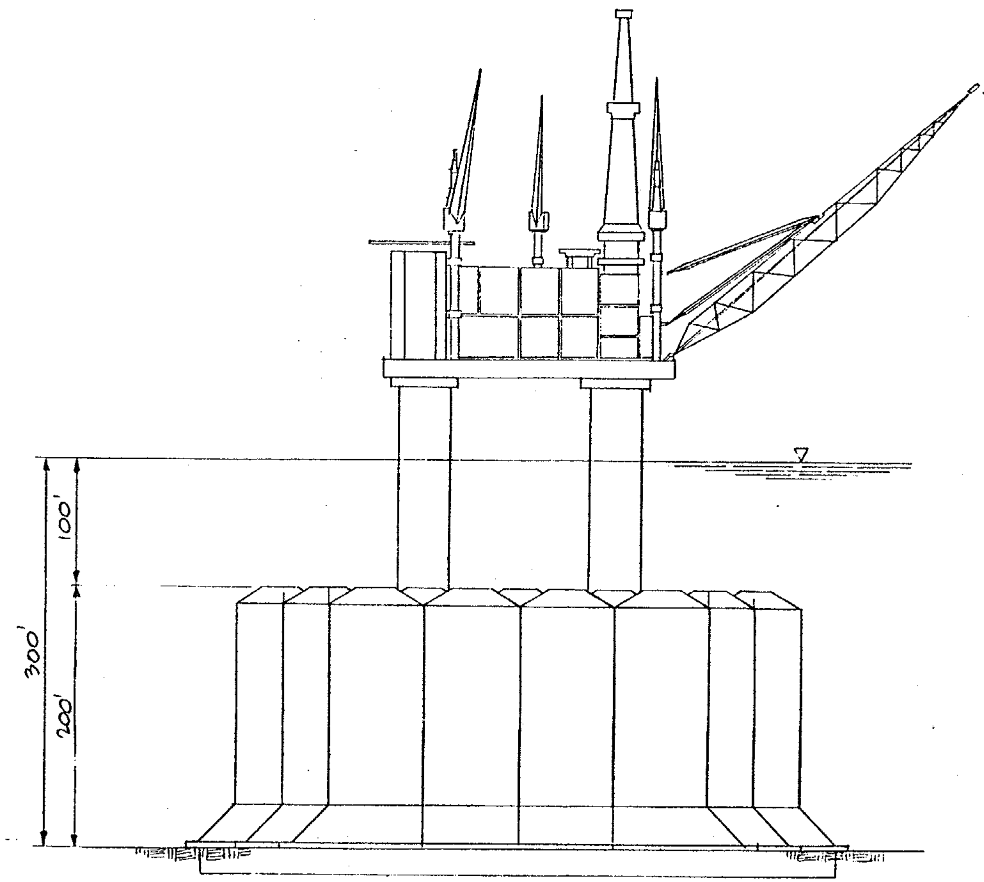
ELEVATION



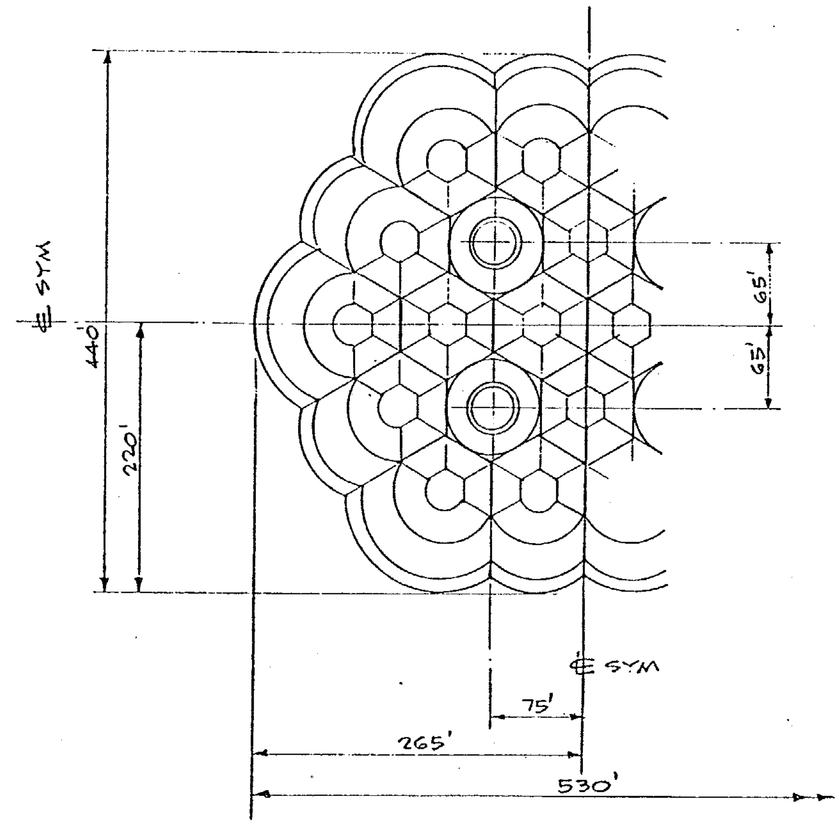
SECTION AT WATERLINE

FIGURE 5-5

CONCRETE GBS
STORAGE CONDITIONS VARIATION
WATER DEPTH = 450 FT
STORAGE CAPACITY = 3,000,000 BBL



ELEVATION



SECTION AT WATERLINE

FIGURE 5-6

CONCRETE GBS
STORAGE CONDITIONS VARIATION
WATER DEPTH = 300 FT.
STORAGE CAPACITY = 2,000,000 BBL

SUMMARY OF CRITICAL SAFETY COEFFICIENTS
SAINT GEORGE BASIN

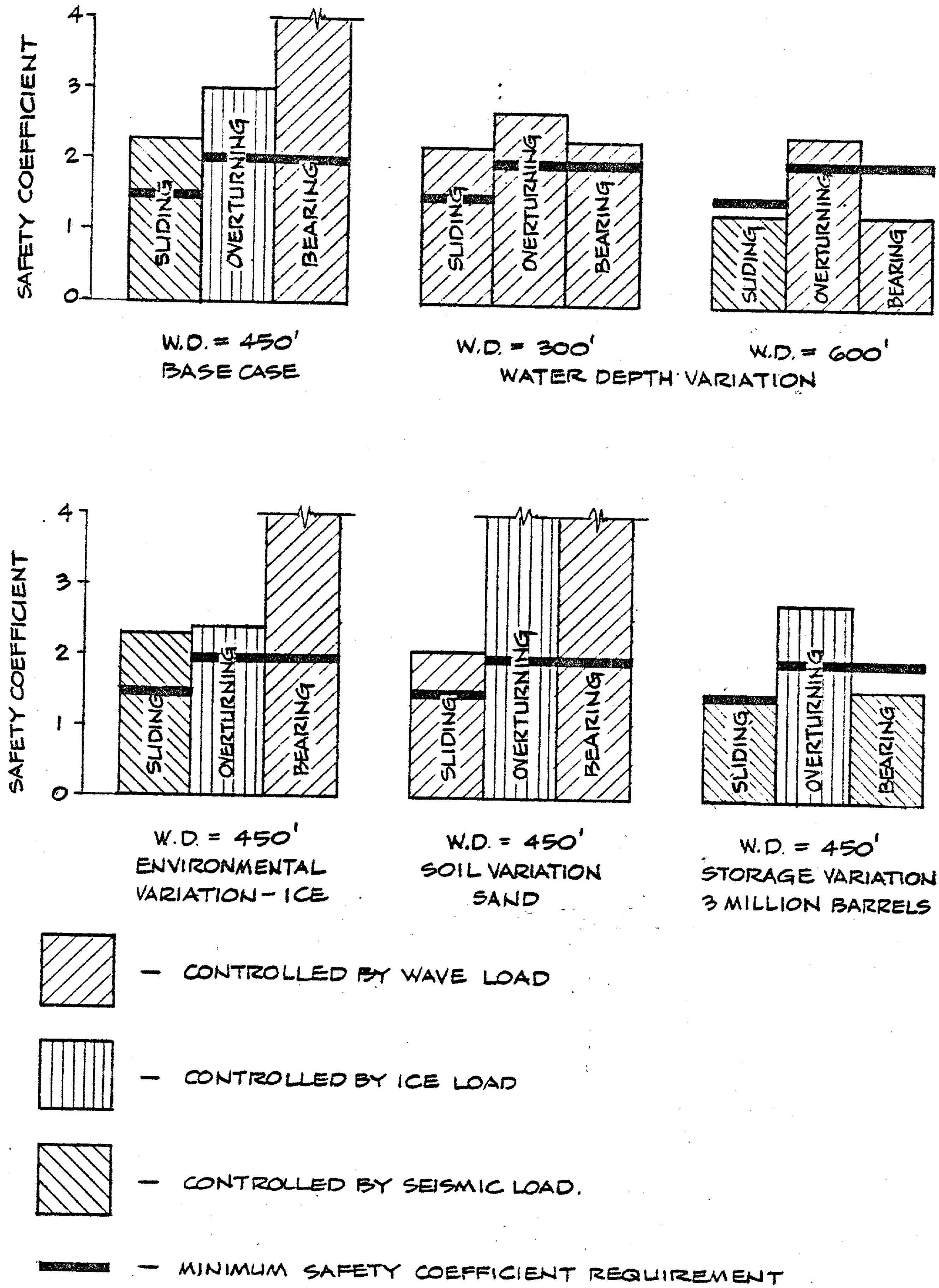
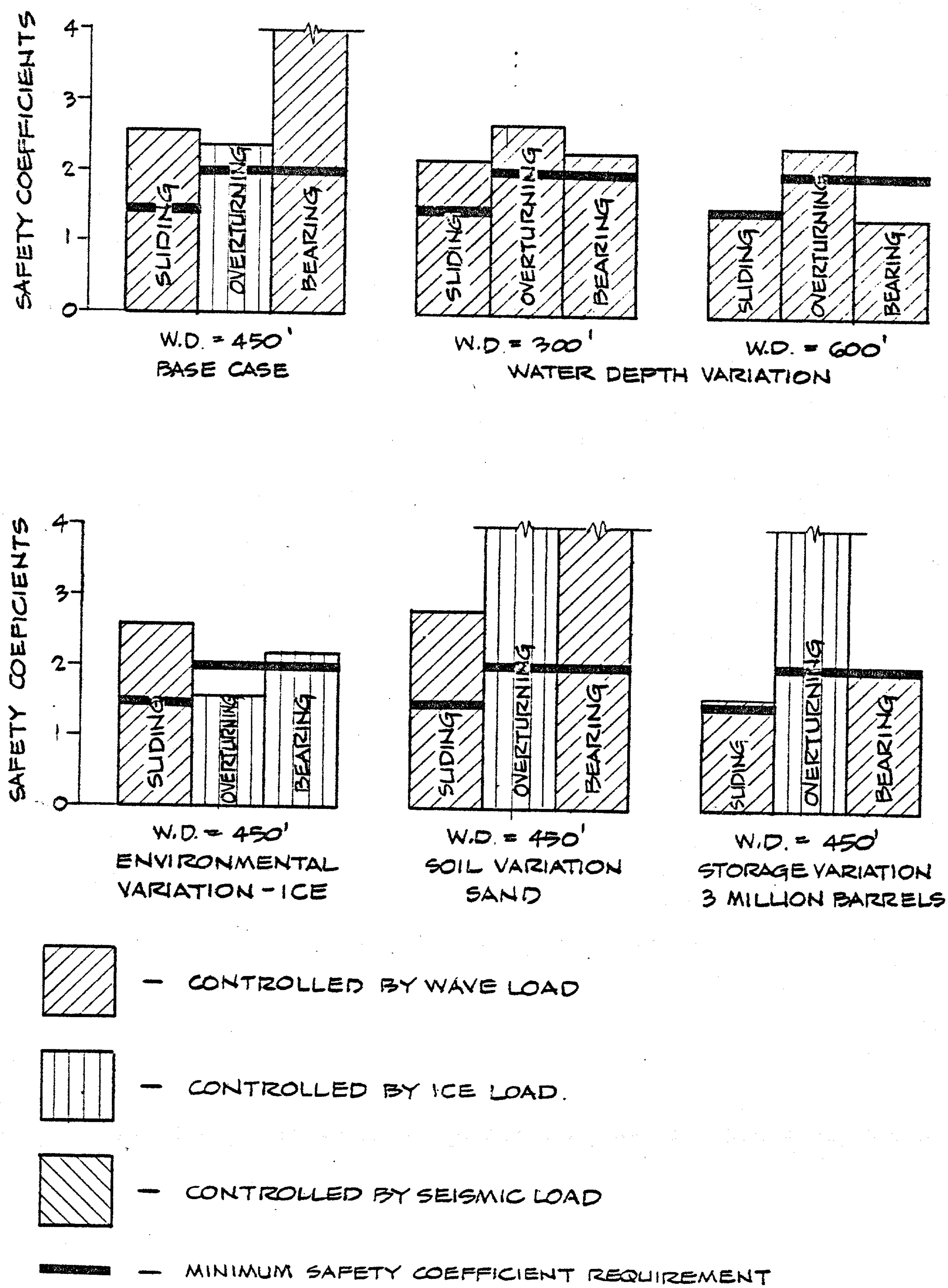


FIGURE 5-7

SUMMARY OF CRITICAL SAFETY COEFFICIENTS
NAVARIN BASIN



SUMMARY OF CRITICAL SAFETY COEFFICIENTS
NORTH ALEUTIAN SHELF

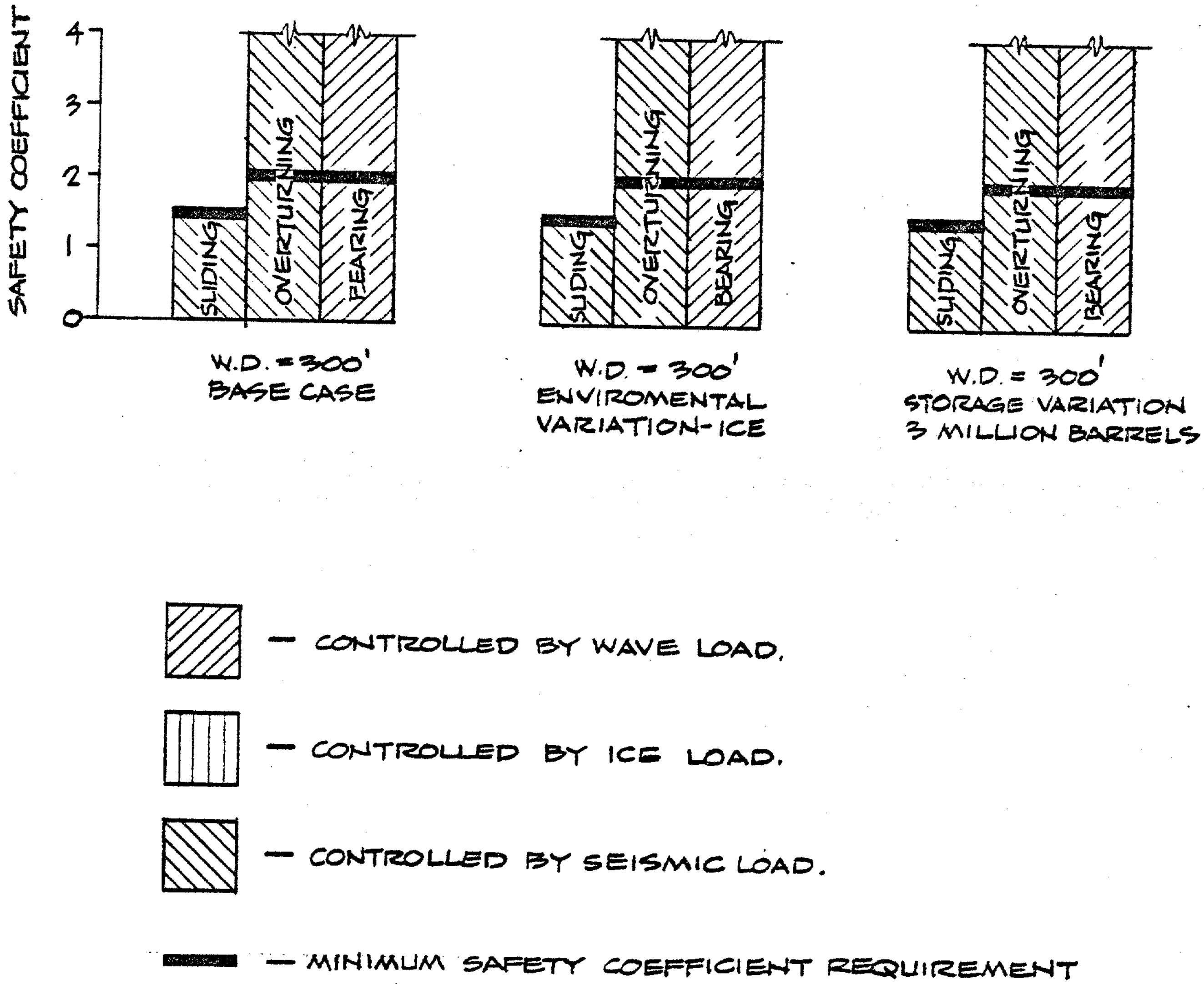
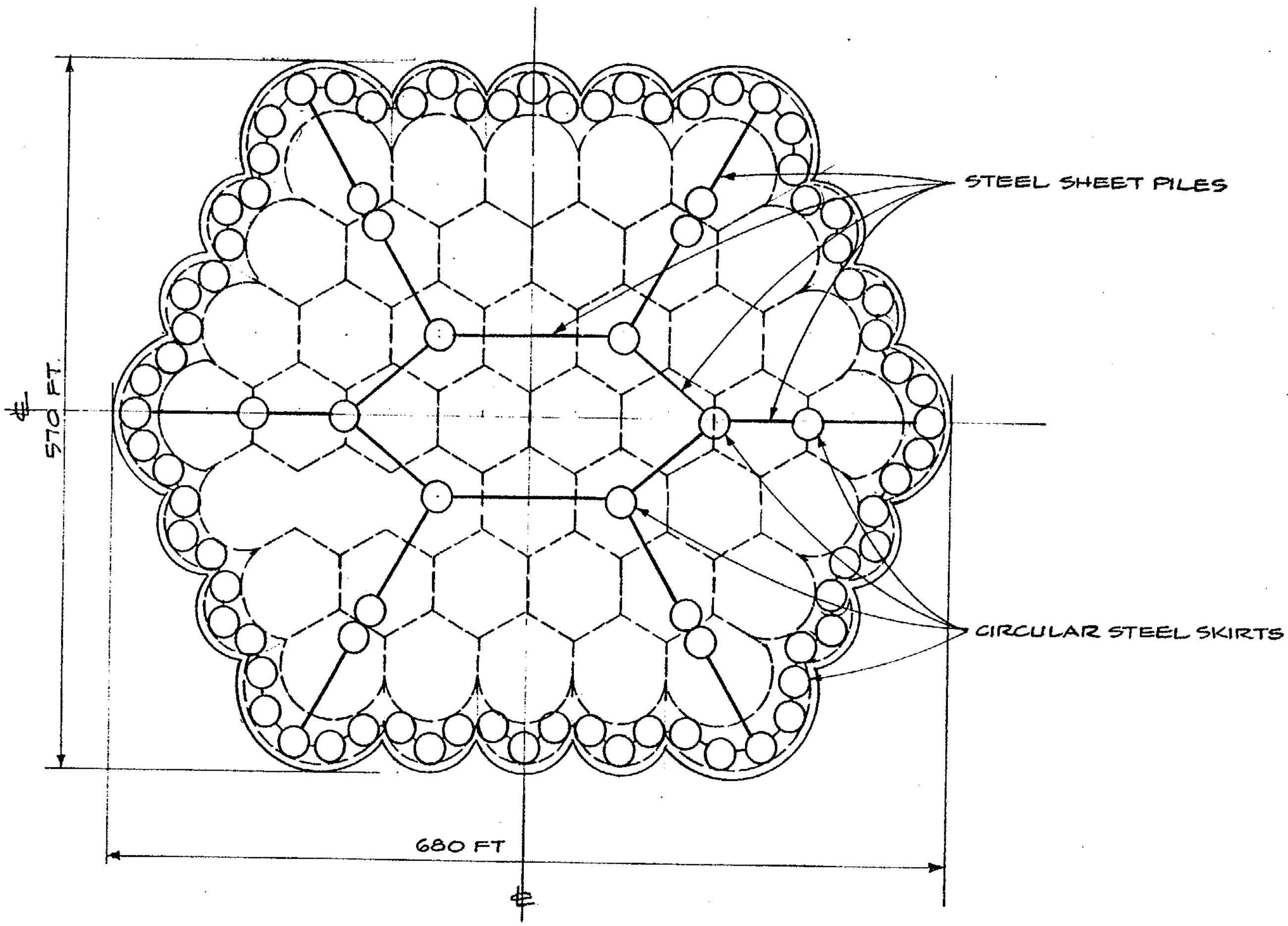
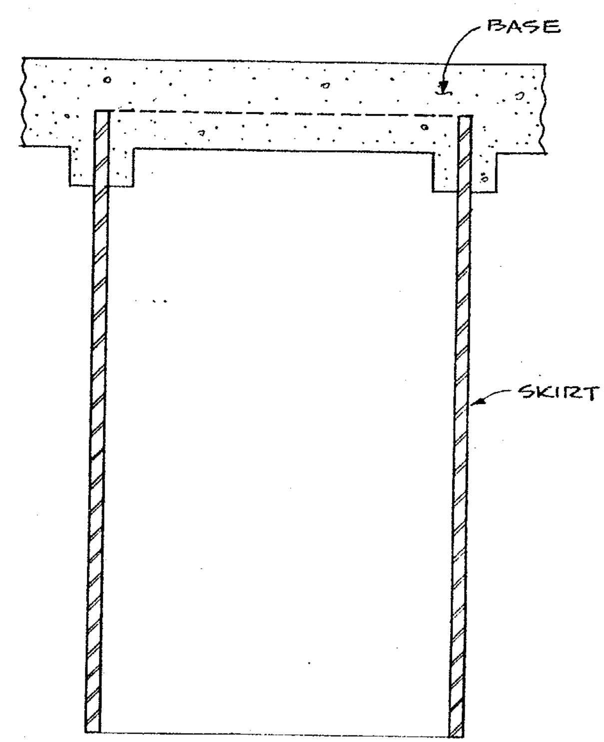


FIGURE 5-9



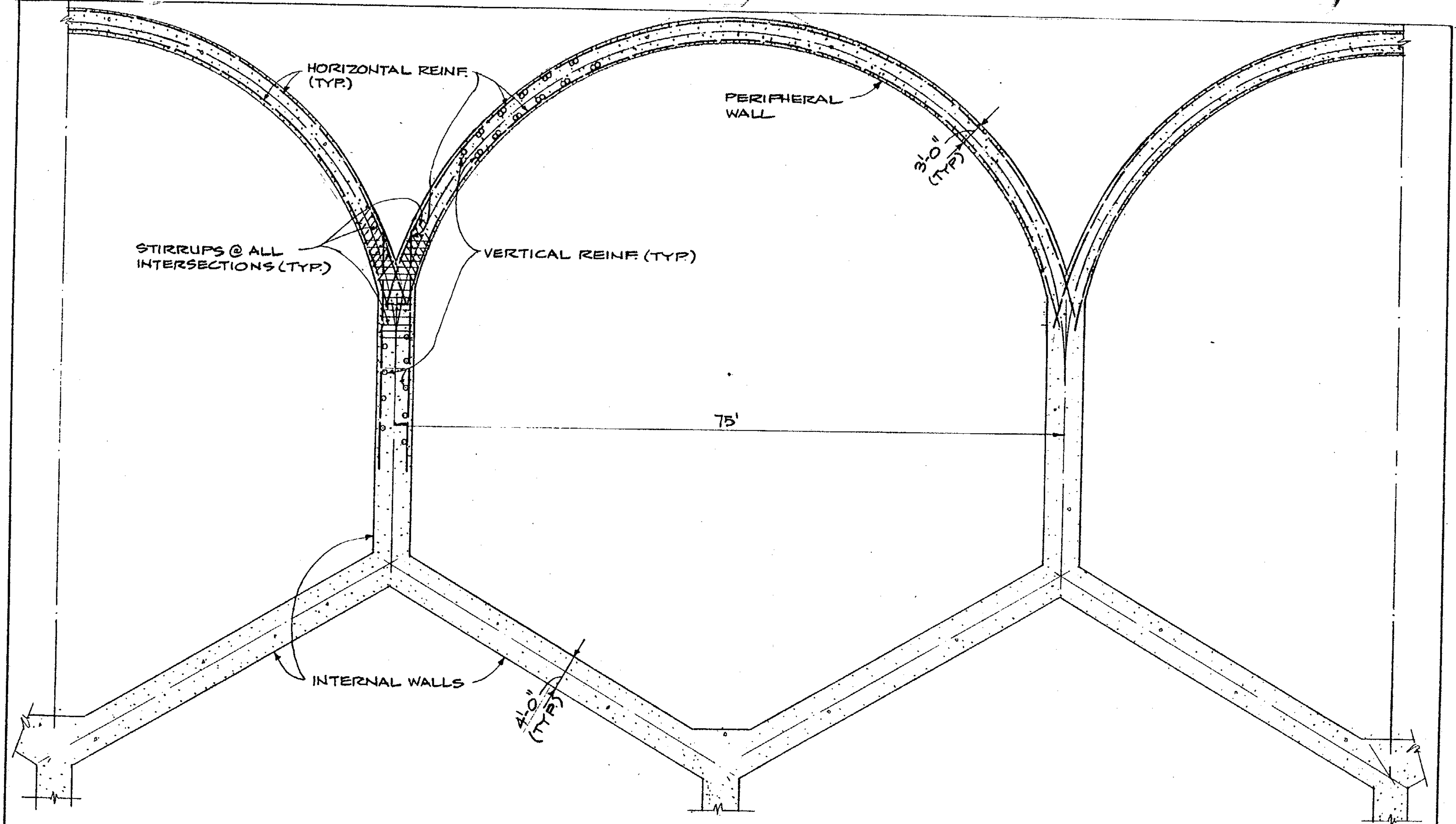
TYPICAL PLAN VIEW OF SKIRTS BENEATH GRAVITY BASE

NOTE: ALL PILES TO EXTEND 20 FT. BELOW CONCRETE.



TYPICAL VERTICAL SECTION AT SKIRT-BASE CONNECTION

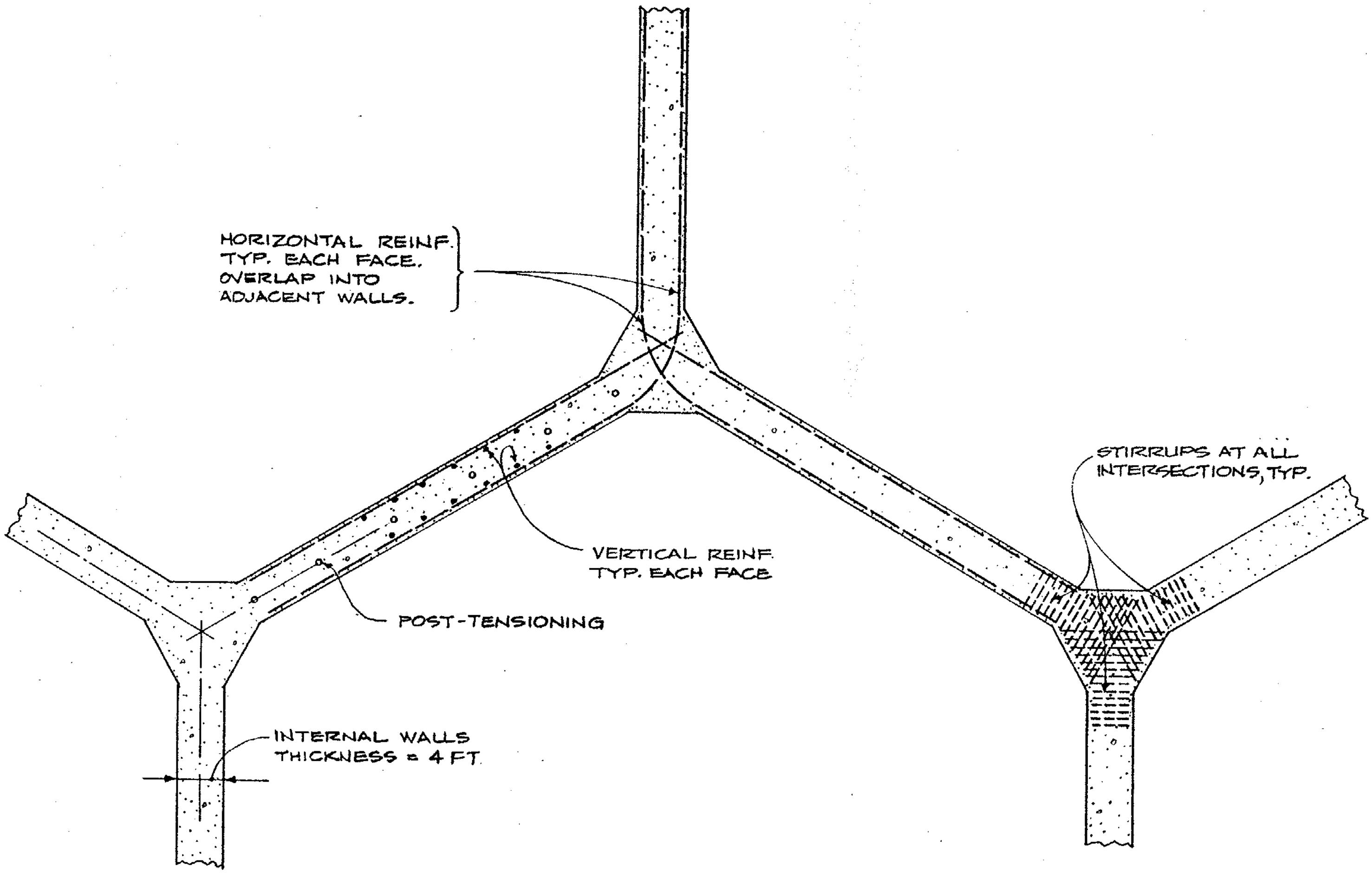
FIGURE 5-10



TYPICAL SECTION AT EXTERIOR CELLS

SCALE: 1" = 10'

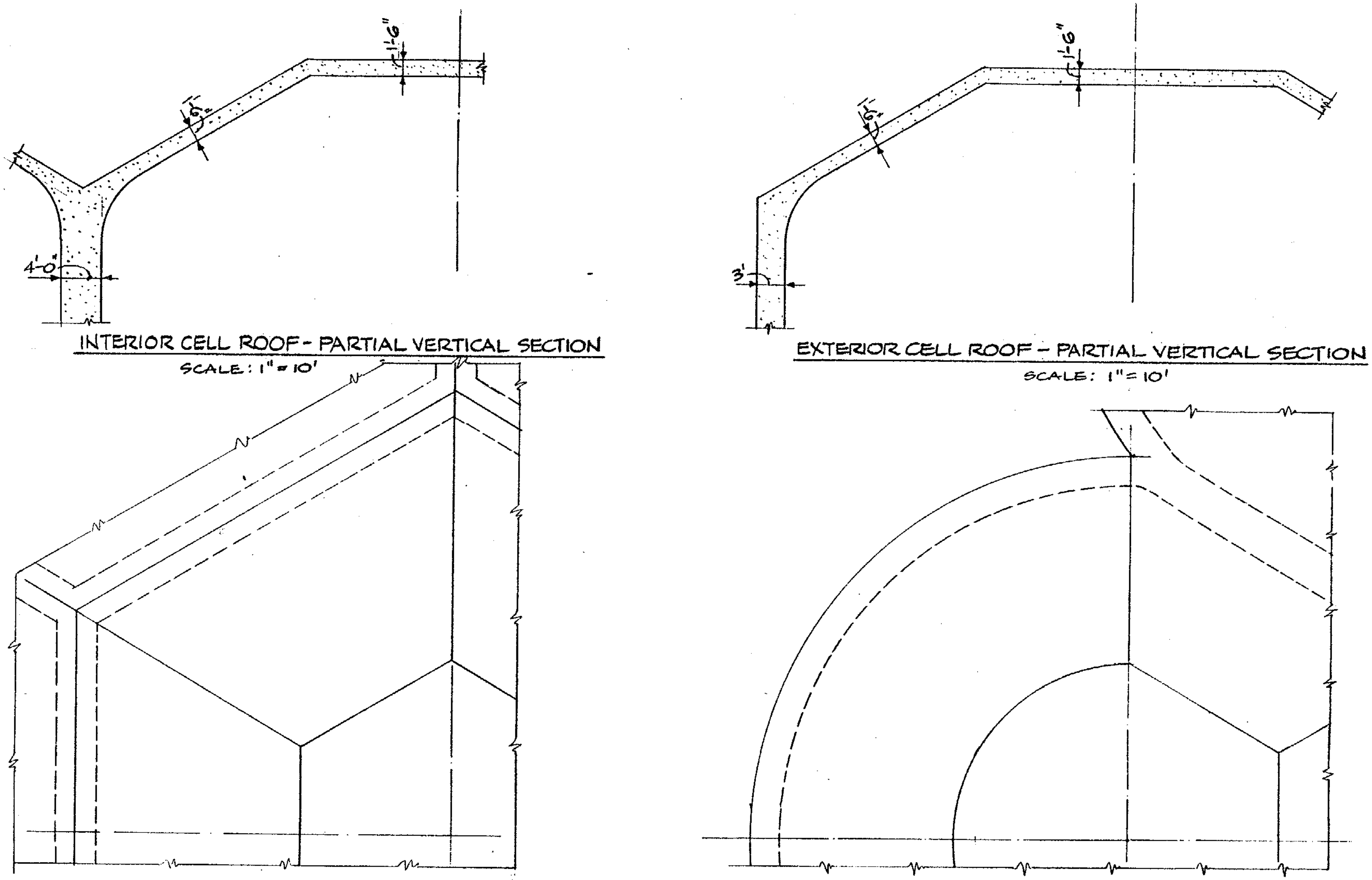
FIGURE 5-11



TYPICAL SECTION AT INTERIOR CELLS

SCALE: 1" = 10'

FIGURE 5-12



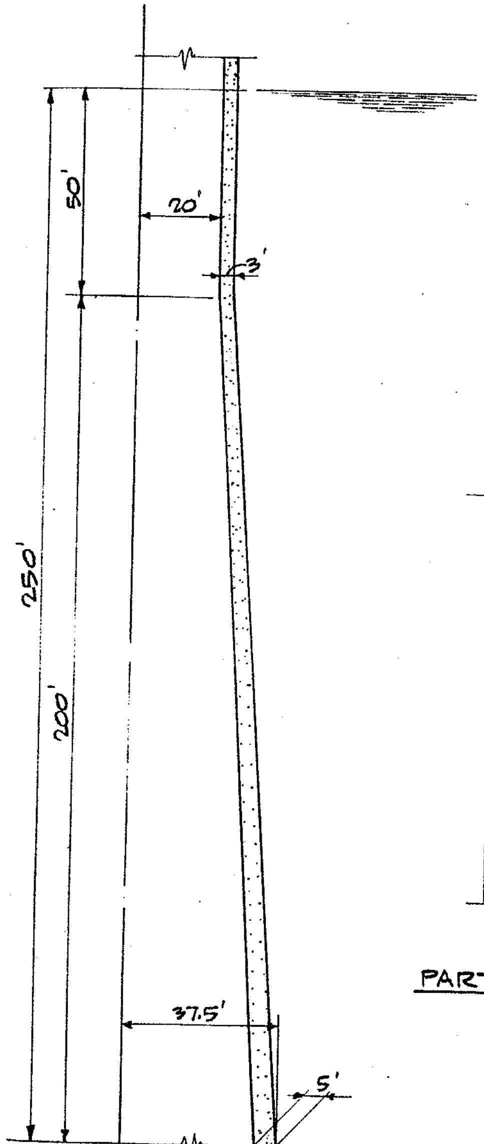
INTERIOR CELL ROOF - PARTIAL VERTICAL SECTION
SCALE: 1" = 10'

EXTERIOR CELL ROOF - PARTIAL VERTICAL SECTION
SCALE: 1" = 10'

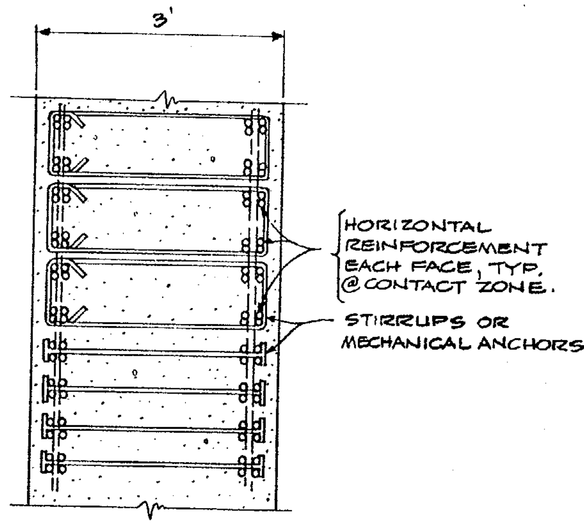
INTERIOR CELL ROOF - PARTIAL PLAN VIEW
SCALE: 1" = 10'

EXTERIOR CELL ROOF - PARTIAL PLAN VIEW

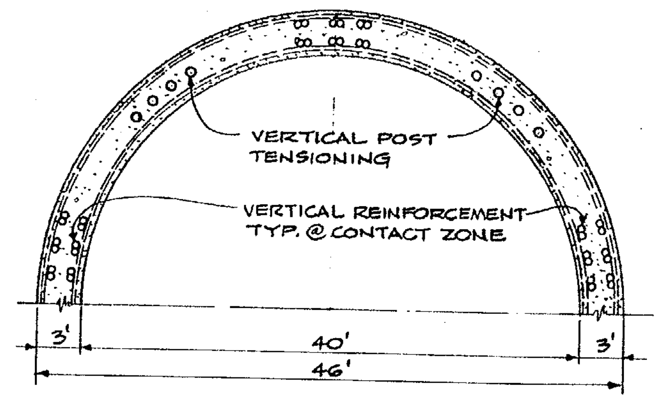
FIGURE 5-13



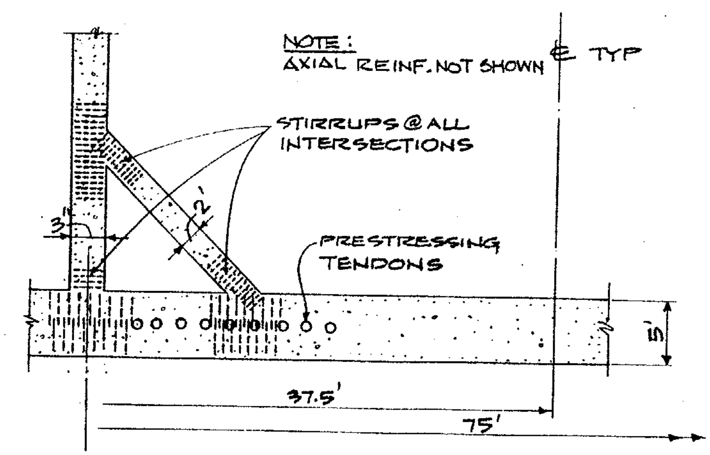
VERTICAL SECTION THROUGH SHAFT
SCALE: 1"=30'



PARTIAL VERTICAL SECTION THROUGH WALL
SCALE: 1"= 2'



PARTIAL HORIZONTAL SECTION THROUGH WALL
SCALE: 1"=10'



PARTIAL SECTION AT WALL-BASE CONNECTION
SCALE: 1"=10'

FIGURE 5-14

TYPICAL SECTIONS AT SHAFT

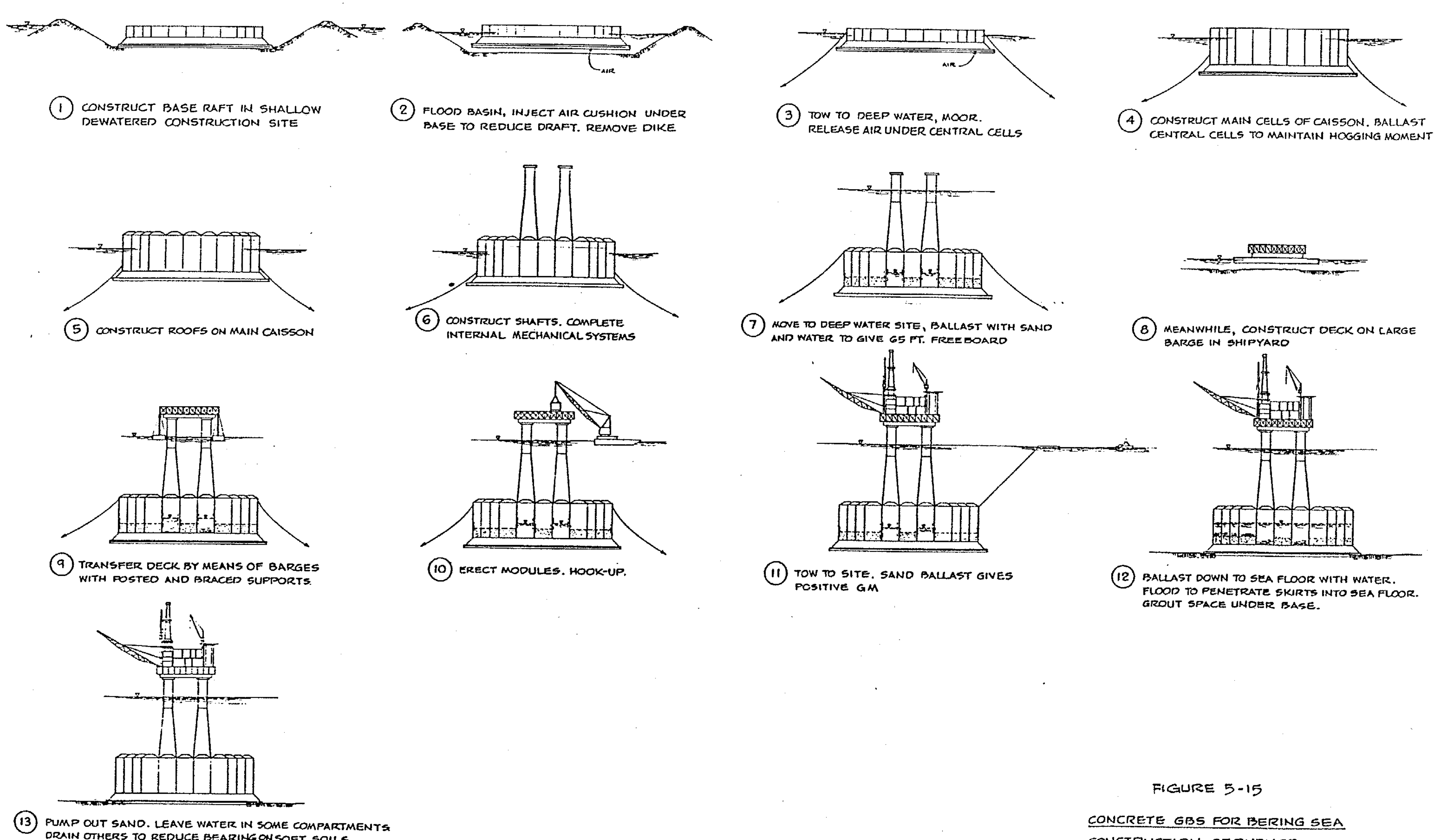
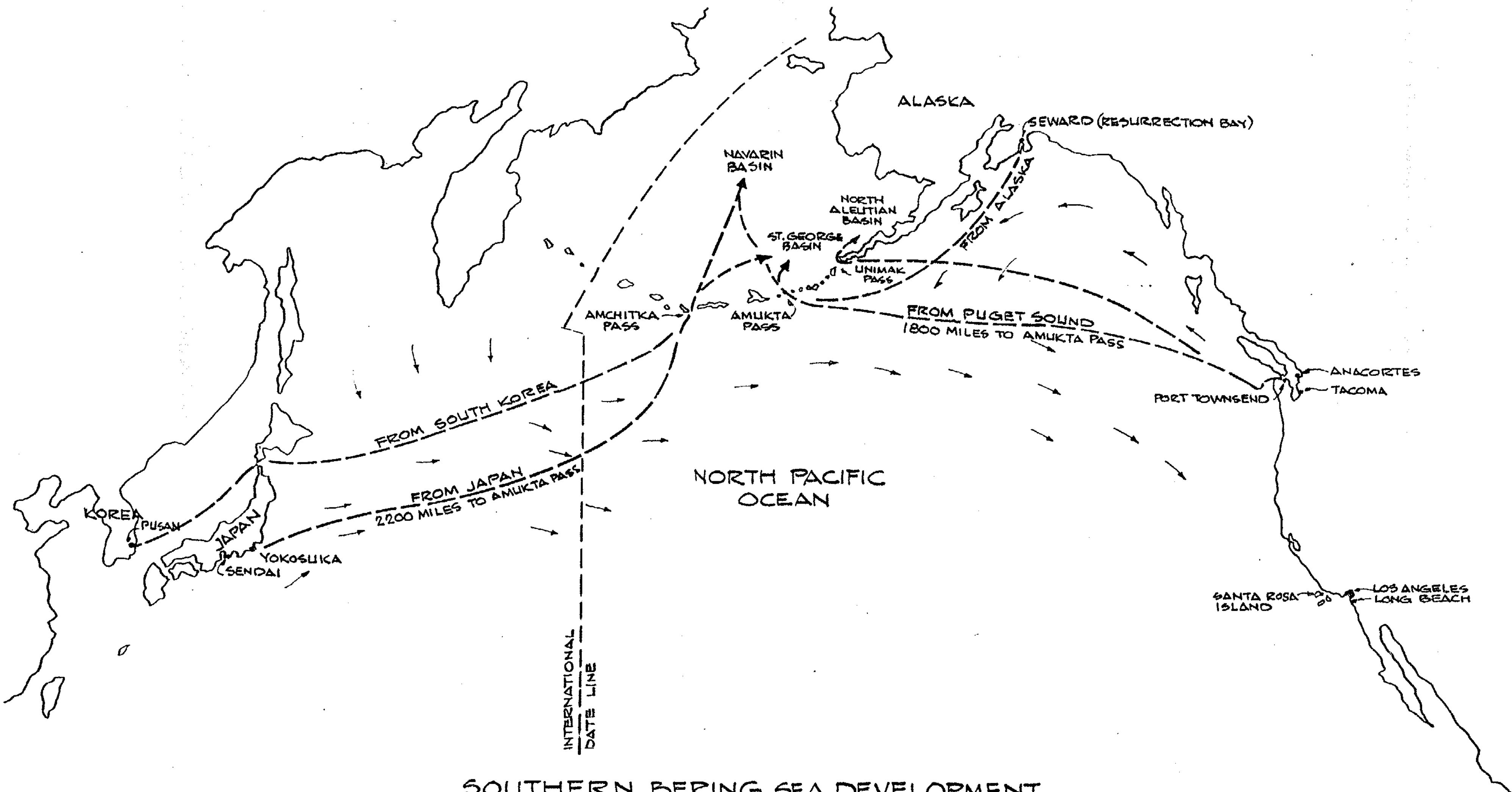


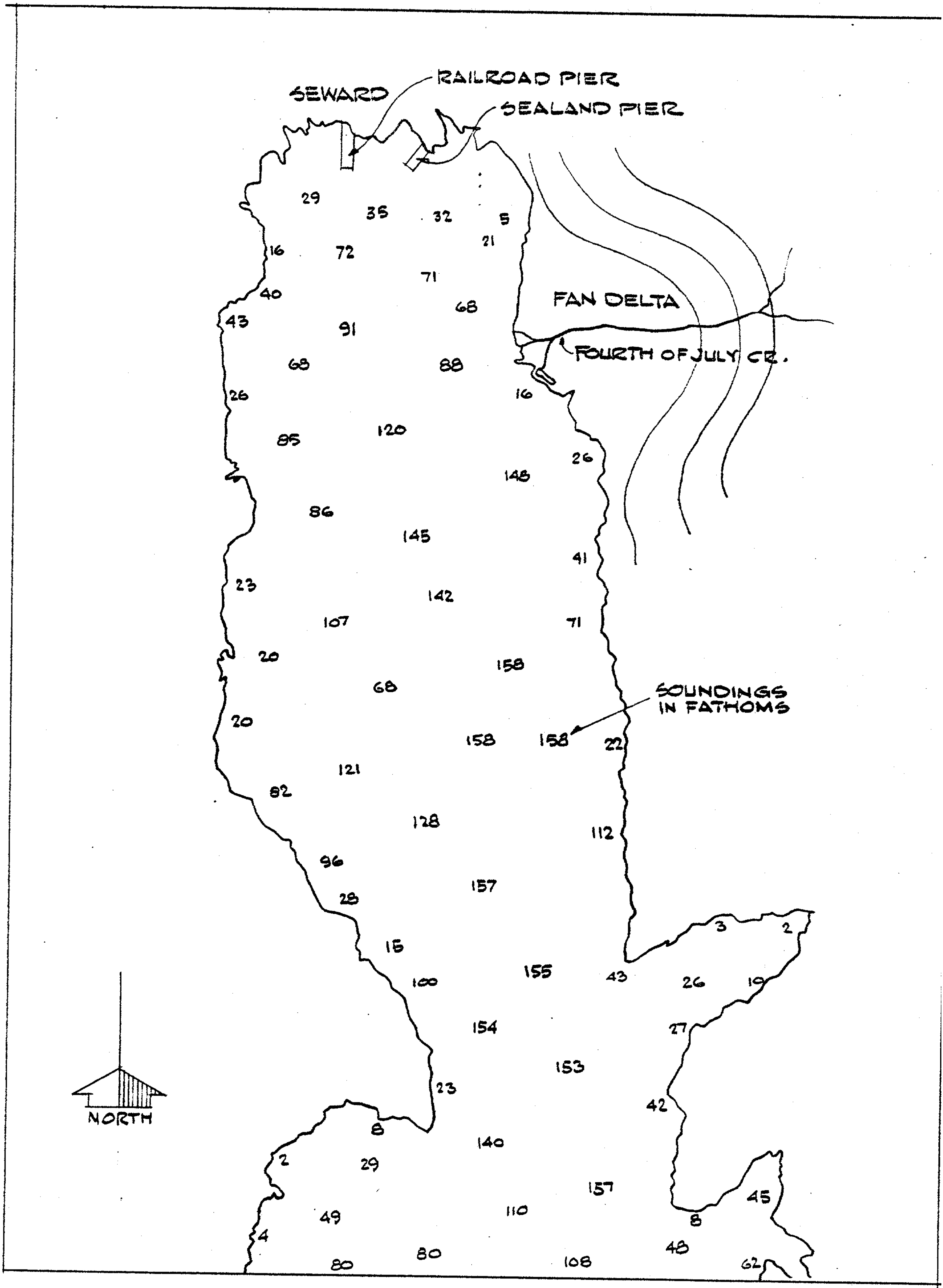
FIGURE 5-15

CONCRETE GBS FOR BERING SEA CONSTRUCTION SEQUENCE



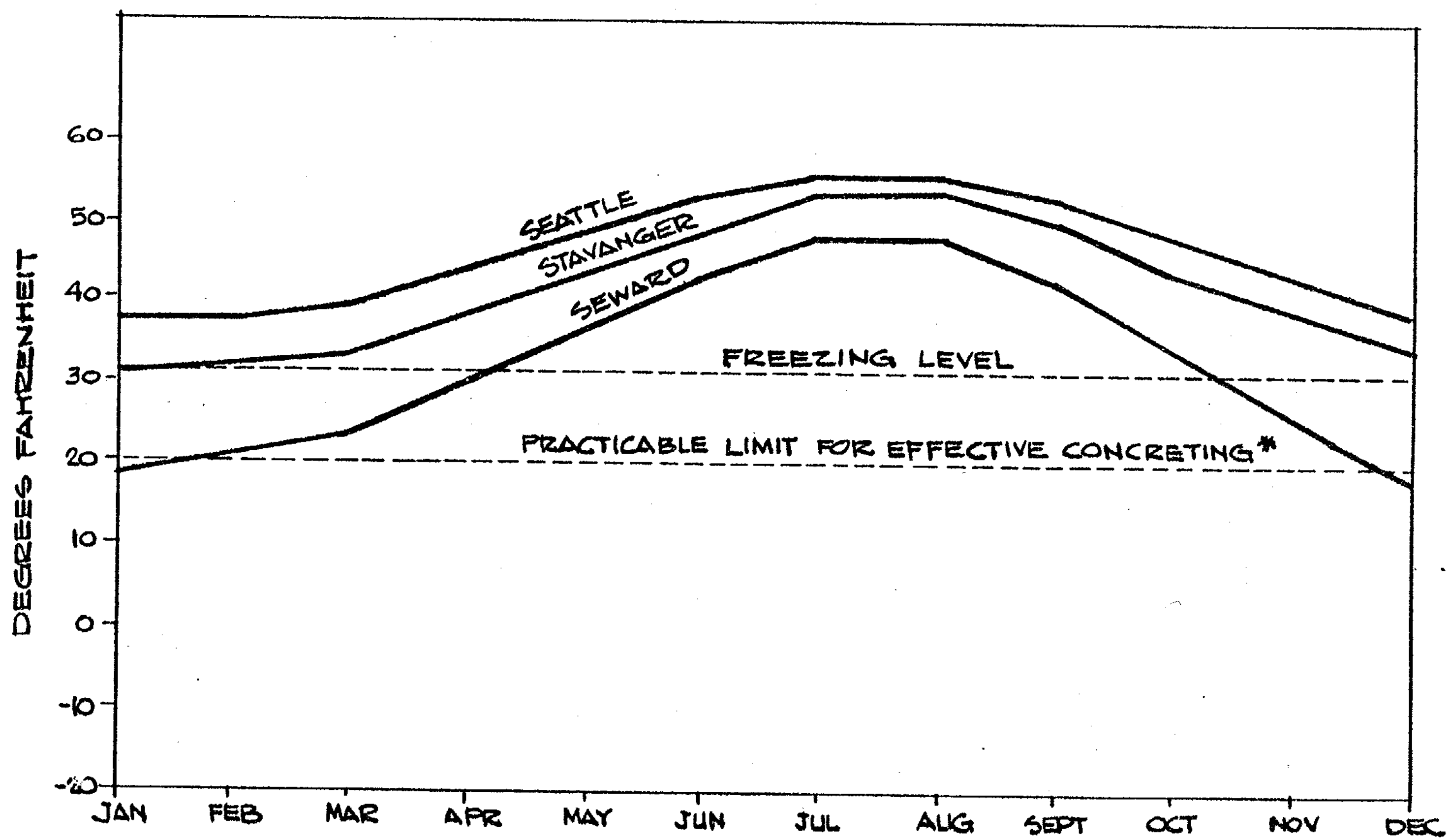
SOUTHERN BERING SEA DEVELOPMENT
CONSTRUCTION SITES AND TOWING ROUTES

FIGURE 5-16



RESURRECTION BAY (SEWARD) ALASKA

FIGURE 5-17



* ASSUMING LOW WIND

COMPARISON OF TEMPERATURE MINIMA FOR SEATTLE, STAVANGER & SEWARD

FIGURE 5-18

TABLE 5-1

ASSUMED MATERIAL PROPERTIES

Material	Property		Quantity	Comments
Normal Weight Concrete	Unit Weight	(p)	155 pcf	Including Reinforcemen
Normal Weight Concrete	Compressive Strength	(f'c)	6000 psi	Cylinder at 28 days
Light Weight Concrete	Unit Weight	(p)	125 pcf	Including Reinforcemen
Light Weight Concrete	Compressive Strength	(f'c)	6000 psi	Cylinder at 28 days
Mild Steel	Yield Strength	(fy)	60000 psi	
Prestressing Tendons	Ultimate Strength	(fpu)	160000 psi	
restressing Tendons	Yield Strength	(fpy)	128000 psi	U.L.S. Method
Prestressing Tendons	Service Strength	(fps)	96000 psi	S.L.S. Method
Sea Water	Unit Weight	(p)	64 pcf	Ballast
Sand	Unit Weight	(p)	112 pcf	Ballast
Crude Oil	Unit Weight	(p)	55 pcf	In Storage

TABLE 5-2 BASE CASE AND VARIATIONS

ST. GEORGE BASIN

	BASE CASE	VARIATIONS FROM BASE CASE[1][2]					
		WATER DEPTH		ICE LOADING	PRODUCTION RATE	SOIL CONDITIONS	STORAGE CONDITIONS
DEPTH	450 FT.	300 FT	600 FT.	*	*	*	*
WAVE	H = 85 FT. T = 15 SEC.	H = 75'(3) T = 14 SEC.	*	*	*	*	*
ICE LOADING	85 K/FT.	*	*	100 K/FT.	*	*	*
SEISMIC - PEAK GROUND ACCELERATION	0.125 g	*	*	*	*	*	*
PRODUCTION RATE	300.000 BPD	*	*	*	100000 BPD	*	*
SOIL CONDITIONS	CLAY Su = 1.0 KSF	*	*	*	*	SAND LWR BOUND $\phi = 30°$	*
STORAGE CONDITIONS	1.000.000 BBL	*	*	*	*	*	3.000.000 BBL
STRUCTURE STABLE?	YES	YES	NO	YES	YES	YES	NO
UNSTABLE MODES & CAUSE	—	—	• SLIDING (SEISMIC) • BEARING (WAVE & SEISMIC)	—	—	—	BEARING (SEISMIC)
TOTAL CONCRETE VOL.	228000 CY	204.000 CY	326.000 CY	SAME AS BASE CASE	SAME AS BASE CASE	SAME AS BASE CASE	311.000 CY

* SAME CONDITION AS BASE CASE

NOTES:

1. LISTED COMPONENT HAS BEEN VARIED FROM BASE CASE CONDITION. ALL OTHER BASE CASE CRITERIA REMAIN THE SAME.
2. VARIATIONS IN WAVE AND SEISMIC LOADING WERE NOT STUDIED. STABILITY MODES CONTROLLED BY THESE CONDITIONS FOR THE BASE CASE WERE FOUND TO BE EXTREMELY CONSERVATIVE, HENCE FURTHER INVESTIGATION FOR SENSITIVITY WAS CONSIDERED UNWARRANTED.
3. THESE VALUES WERE USED DUE TO THE ANTICIPATED LOWER WAVE MAGNITUDES IN MORE SHALLOW WATER.

TABLE 5-3 BASE CASE AND VARIATIONS
NAVARIN BASIN

	BASE CASE	VARIATIONS FROM BASE CASE (1) (2)					
		WATER DEPTH		ICE LOADING	PRODUCTION RATE	SOIL CONDITIONS	STORAGE CONDITIONS
DEPTH	450 FT.	300 FT.	600 FT.	*	*	*	*
WAVE	H = 85 FT. T = 15 SEC.	H = 75' (3) T = 14 SEC.	*	*	*	*	*
ICE LOADING	110 K/FT.	*	*	170 K/FT.	*	*	*
SEISMIC - PEAK GROUND ACCELERATION	0.05 g	*	*	*	*	*	*
PRODUCTION RATE	300.000 BPD	*	*	*	100.000 BPD	*	*
SOIL CONDITIONS	CLAY Su = 1.0 KSF	*	*	*	*	SAND LWR BOUND $\phi = 30°$	*
STORAGE CONDITIONS	1.000.000 BBL	*	*	*	*	*	3.000.000 BBL
STRUCTURE STABLE?	YES	YES	NO	NO	YES	YES	YES
UNSTABLE MODES & CAUSE	—	—	BEARING (WAVE)	OVERTURNING (WAVE)	—	—	—
TOTAL CONCRETE VOL.	228.000 CY	204.000 CY	326.000 CY	SAME AS BASE CASE	SAME AS BASE CASE	SAME AS BASE CASE	311.000 CY

* SAME CONDITION AS BASE CASE

NOTES:
1. LISTED COMPONENT HAS BEEN VARIED FROM BASE CASE CONDITION. ALL OTHER BASE CASE CRITERIA REMAIN THE SAME.
2. VARIATIONS IN WAVE AND SEISMIC LOADING WERE NOT STUDIED. STABILITY MODES CONTROLLED BY THESE CONDITIONS FOR THE BASE CASE WERE FOUND TO BE EXTREMELY CONSERVATIVE, HENCE FURTHER INVESTIGATION FOR SENSITIVITY WAS CONSIDERED
3. THESE VALUES WERE USED DUE TO THE ANTICIPATED LOWER WAVE MAGNITUDES IN MORE SHALLOW WATER.

TABLE 5-4 BASE CASE AND VARIATIONS

NORTH ALEUTIAN SHELF

	BASE CASE	VARIATIONS FROM BASE CASE (1)(2)(3)				
		WATER DEPTH		ICE LOADING	PRODUCTION RATE	STORAGE CONDITIONS
DEPTH	300 FT.	200 FT.	350 FT.	*	*	*
WAVE	H = 75 FT. T = 14 SEC.	*	*	*	*	*
ICE LOADING	85 K/FT.	*	*	110 K/FT.	*	*
SEISMIC - PEAK GROUND ACCELERATION	0.24g	*	*	*	*	*
PRODUCTION RATE	300.000 BPD.	*	*	*	100.000 BPD	*
SOIL CONDITIONS	SAND $\phi = 30°$	*	*	*	*	*
STORAGE CONDITIONS	1.000.000 BBL	*	*	*	*	2000 000 BBL
STRUCTURE STABLE?	YES	YES	YES	YES	YES	YES
UNSTABLE MODES & CAUSE	—	—	—	—	—	—
TOTAL CONCRETE VOL.	138.000 CY	(4)	(4)	SAME AS BASE CASE	SAME AS BASE CASE	190.000 CY

* SAME CONDITION AS BASE CASE

NOTES:

1. LISTED COMPONENT HAS BEEN VARIED FROM BASE CASE CONDITION. ALL OTHER BASE CASE CRITERIA REMAIN THE SAME.
2. VARIATIONS IN WAVE AND SEISMIC LOADING WERE NOT STUDIED. STABILITY MODES CONTROLLED BY THESE CONDITIONS FOR THE BASE CASE WERE FOUND TO BE EXTREMELY CONSERVATIVE, HENCE FURTHER INVESTIGATION FOR SENSITIVITY WAS CONSIDERED [illegible]
3. NO VARIATION IN THE SOIL CONDITIONS WERE CONSIDERED BECAUSE ONLY GRANULAR SOIL WAS IDENTIFIED IN THE DESIGN CRITERIA FOR THIS BASIN.
4. STRUCTURES WERE NOT DIRECTLY DEVELOPED FOR THESE CASES, HOWEVER IT IS CONCLUDED THAT GRAVITY STRUCTURES CAN BE INSTALLED AT THESE DEPTHS. SEE SECTION 5.52 FOR DISCUSSION.

TABLE OF CONTENTS

SECTION 6.0

LIST OF FIGURES

SECTION 6.0

LIST OF TABLES

SECTION 6.0

6.0 COST ESTIMATES AND SCHEDULES - PLATFORMS

6.1 General

6.1.1 Scope of Work

The cost estimates presented in this section are based on the preliminary designs which are presented in this report. Included are cost estimates for the following items:

- Drilling and Production Topside Facilities
- Steel Pile Supported Structures
- Concrete Gravity Structure

Also included are cost sensitivities for key parameters which may vary from the design basis conditions. These include:

- Water Depth
- Production Rate
- Environmental Condition
- Soil Conditions

Note: Costs for pipeline, shore terminal and offshore storage and loading structure are given in Section 10.

6.1.2 Objectives

This cost estimate has been presented in such a way that the following objectives will be met as much as possible within the limited framework of the project.

- The assumptions or background sources of information are clearly identified.
- The cost estimating methods and basis of each major cost item is identified.
- This data base will provide a means for future cost estimates, tailored to the requirements of individual operating companies.

6.1.3 Date of Estimate and Escalation

The estimated costs are based on June 1982 costs in U. S. Dollars. No escalation beyond this time has been included.

6.1.4 Accuracy of Estimate and Cost Estimating Allowance

The cost estimates included herein are thought to be accurate to within plus or minus 20 to 30 percent. A cost estimating allowance of 30 percent has been included in all of the cost summary sheets in this section. It is identified on each summary work sheet as such. It is recommended that this allowance be carefully reviewed to ensure that it is consistent with the level of confidence assigned to both the design basis conditions and the cost estimating bases and methods.

6.1.5 Home Office Costs

The summary tables include an allowance of 10 percent for home office costs. This allowance is intended to cover the following items:

- Design engineering.
- Project management.

- Site investigations
- Purchasing and expediting

6.2 Drilling and Production Topside Facilities

6.2.1 Items Included

Topside facilities are assumed to be fabricated as separate modules whether set on a steel jacket at the site of production or on a concrete gravity base nearshore for tow-out of the platform as a completed unit. The estimates include all material (including structural steel) equipment and labor costs associated with the fabrication of the modules.

These estimates do not include the following items which are shown elsewhere in the cost estimates:

- Transportation from the fabrication yard
- Setting of the modules on the support structure
- The steel structure of the module support deck
- Any of the equipment or materials in the utility shaft of a gravity base structure having subsea storage

Labor costs are included for:

- All hookup offshore in the case of a steel structure.
- Hookup nearshore in the case of a concrete gravity structure with hookup completion and commissioning offshore. In this case, it is assumed hookup is 90% completed nearshore. The remaining 10% is done offshore together with a manhour allowance for de-commissioning the platform after the sea voyage and removal of sea fastenings.

A labor cost allowance is made for the specific case of setting modules offshore which includes:

- Assistance in module setting, skidding and tie-down
- Life-support systems hookup to make the platform habitable for the main hookup crew.

6.2.2 Basis of Cost Estimate

6.2.2.1 Data Sources

The cost estimates for the several categories of topside components were factored from recent and historical data on North Sea platform projects. The smaller case of crude oil throughput, 100 MBPD, was factored from steel-structure platforms like Beryl B but mostly from Murchison, since their production rates are similar, being about 120 MBPD to 150 MBPD.

The components of the larger case of throughput in this study, 300 MBPD, were factored from concrete gravity base structures like Statfjord A and the Golden Block, 34/10, platform now in the design stage; again, because the planned production rates of these platforms are closer to the larger case of throughput. Guidance was also provided by cost data from other concrete gravity base structures like Statfjord B, now in place offshore, and Statfjord C which is under construction.

6.2.2.2 Module Fabrication Costs

Fabrication costs developed from the data base are characteristic of European construction. These were adjusted to reflect expected Korean/Japanese fabrication costs.

6.2.2.3 Factoring Principles

Topside-component cost estimation is divided into seven categories:

- Quarters and helideck
- Drilling
- Production
- Power generation
- Utilities
- Flareboom
- Other equipment

Factoring principles are as follows:

- o Quarters, minus the price of the helideck, are cost estimated on a cost-per-bed basis.
- o Drilling equipment, including the complete derrick, crown block, rotary table, draw works, mud and cement storage, de-sanders, shale shakers, logging laboratory and pipe racks are factored on a per rig basis.
- o Production modules and facilities are factored on the basis of production throughput and include well heads, gas compression and injection, water injection and treatment, separation and mainline pumps.

- Power generation including emergency-power generation is factored on the basis of U.S. dollars per megawatt, but only on the presumption that the data base, from which such factoring is made, must be near the power production levels chosen for the platforms conceptually developed in this study.
- Utilities cost estimates are factored on the basis of the maximum number of personnel to be accommodated on the platform with an allowance for the severity of the weather in the Bering Sea. This is particularly important in the Bering Sea where all working and accommodation spaces are fully enclosed.
- Flareboom costs are factored on the basis of production throughput.
- Other equipment includes:
  - Management services in fabrication yards
  - Survival equipment
  - Cranes
  - Navigational aids
  - Vendor representatives
  - Bumpers and guides
  - Fire protection
  - Communications
  - Computer and audit services
  - Welding institute and fabricator consultation
  - Onsite engineering services
  - Sea fastening as applicable

6.2.2.4 Labor

Productivity - Labor productivity and skill/specialty availability is assumed to be the same for the Bering Sea as for the North Sea.

Labor Rates - Labor rates are separated into three categories:

- Labor offshore on or pertaining to work on the platform at sea.
- Labor nearshore with shipyard facilities and no camp or subsistence costs.
- Labor nearshore in a remote environment.

The last applies to hookup and interfacing of topside modules with a concrete structure having storage in its base. The remote location is one where quiet water is deep enough to lower the concrete structure for mating with the topsides and deck structure. There, work of a hookup nature is needed to complete the topsides/support structure mating and component interfacing with equipment in the utility shaft before tow-out.

The following rates are taken for each of the three labor categories -

Offshore -

\$55 per hour: average craft rate incl. uplifts/premiums

\$20 per hour: indirects, overhead and profit at about 40%

\$10 per hour: catering allowance per manhour offshore

\$85 per manhour total, assuming 7-12 hour shifts per man per week.

Nearshore/Shipyard - applies to module hookup on a steel deck for a concrete gravity base structure. \$55 per manhour is used. This is based on recent San Francisco Bay area experience and includes contractor's indirects, overhead and profit.

Nearshore/Remote Location-

$ 50 per hour: average rate incl. uplifts/premiums

$ 70 per hour: indirects* (at 140%)

$ 20 per hour: camp/catering

$140 per manhour: direct manual labor rate

This manhour rate only applies to mating and interface connections between topsides and utility shaft components at a remote, deep draft location. If this remote location is in Alaska, an Alaska use tax must also be added (see Section 10.3.2.3).

* Field indirects include temporary construction facilities, construction equipment, tools and consumables, miscellaneous construction services and field office costs. The $70/hour camp cost represents an upper bound condition for a very remote site. If sites such as Port Townsend or Port Angeles, Washington, or the Inland Sea of Japan are used, this cost could be reduced substantially.

6.2.3 Order of Magnitude Capital Costs

6.2.3.1 Estimated Topside-Module Equipment/Fabrication Costs

The equipment/fabrication costs for 100 MBPD and 300 MBPD topsides are given Table 6-1. No distinction is made in the materials cost between steel and concrete support structures. Concrete structure topsides can be built in two alternatives:

- Pipeline to shore with mainline pumps included in the topsides.
- Local subsea oil storage for offshore tanker loading.

The latter alternative requires treatment of the storage-tank ballast water on the platform instead of the mainline pumps, but no account has been taken of cost differences between these facilities.

6.2.3.2 Manhour Costs - Hookup

Hookup manhours are estimated from historical data on North Sea experience. There is no inflation or escalation associated with adapting these figures to the Bering Sea, and the productivity in the Bering Sea is assumed to be the same as in the North Sea.

Table 6-2 summarizes hookup costs for the steel and concrete platforms.

6.2.3.3 Transportation and Safety Vessel Costs During Hookup

During the period planned for hookup offshore fixed-wing and helicopter transportation will be needed for crew changes, and supply boats will be used to haul bulk materials from the supply base to the platform.

A safety vessel will stand off the platform at all times.

Estimated cost summaries are given in Table 6 - 3.

6.2.3.4 Total Estimated Topside Costs

Module costs and hookup-labor costs are combined in Tables 6-4 and 6-5 to develop total estimated topside costs. The steel jacket support structure is treated separately from the concrete gravity base support structure for reasons of different manhour rates applicable in the two alternatives and different total transportation costs during offshore hookup.

6.2.3.5 Cash Flow

Cash flow by quarters (excluding transportation costs) is developed in Figures 6-1 and 6-2 for the steel-jacket topside facilities in the 100 MBPD and 300 MBPD cases of throughput.

6.2.4 Operating Costs

A rule of thumb in the North Sea for a 150,000 BPD platform is that the typical operating costs range from 2-1/2% to 7-1/2% of the total capital costs per year. This applies over the first 5 to 10 years.

As an example, a 150,000 BPD platform may experience operating expenses of 50 million U.S. dollars per year which includes:

- Wages
- Structure
- Equipment
- Well workover
- Marine transport
- Pipelines
- All risks insurance

but does not include drilling.

Insurance alone accounts for 25% to 33-1/3% of the total.

These cost indications are based on offshore platforms approximately 150 miles from the onshore base.

In application to the Bering Sea, operating expenses are expected to be increased in several categories:

<u>Insurance</u> - Even if the percentage range of the total operating expense remains unchanged, the insurance cost will increase due to the increased capital cost of the Bering Sea facilities.

<u>Transportation</u> - In the capital cost estimations, a platform in the Navarin Basin is expected to accrue 60 million dollars per year in air/sea transportation and operation of the safety ship alone during the topside hookup phase. The annual costs for air transportation and the safety vessel will remain the same during the operating phase of platform activity as in the hookup phase, but supply boat expenses can be reduced. The reduction is based on the assumptions of long-term supply boat leasing and reduced fuel costs (Section 10.3.5.2).

<u>Pipelines</u> - Pipelines are included in an unknown percentage of the total operating expense in the example given above. Operating expenses for the pipelines in the Bering Sea are discussed in a separate section of this report, but no clear comparisons can be made between pipeline operating costs in the North Sea and the Bering Sea. Nevertheless, the St. George and Navarin pipelines are much longer than the 150 (statute) miles taken as the norm in the North Sea example. Increased pipeline operating costs can be expected.

For broad estimation purposes, an operating cost per year of up to 7-1/2% of the total capital cost may be used. The costs of transportation should be added separately.

6.3 Steel Structure

6.3.1 Items Included

This estimate contains all material, equipment and labor costs associated with fabrication, transportation and installation of the following major components:

- Jacket (including deck support girders but not equipment module steel)
- Piling
- Conductors

In addition, the cost of the following supplemental items is included:

- Mobilization of derrick barge and launch barge spreads.
- The offshore setting (but not hookup) of topside modules. This was included in this section because it is just one of the many derrick barge activities.
- The transportation of the topside modules from the fabrication yard to the offshore construction site.

6.3.2 Basis of Cost Estimate

The costs included in this section were built up principally on a unit cost basis. The quantities were derived from the details contained in Section 4 of this report. The unit costs were based on the collective experience of the project team, and the contributions of the following organizations:

Kaiser Steel Corporation - Ivor Thomas, 707/257-5000

- Unit cost per ton for fabrication and loadout of jacket, piling and conductors.

McDermott Incorporated - Byron J. Eiermann, 504/587-4411

- Derrick barge spread costs per day
- Mobilization costs

Crowley Maritime - Carl Stubbs, 206/583-8100

- Towing spread costs
- Towing time estimates

6.3.3 Structural Fabrication

The cost for the structural fabrication has been based on either West Coast of the U. S. or Japan as the fabrication sites. The following unit rates have been assigned to this work:

	Material Cost $/Ton	Fabrication Cost $/Ton	Total Cost $/Ton
Jacket*	800	4300	5500
Piling	700	1100	1800
Conductors	700	900	1600

*The cost of all jacket appurtenances has been included in this item.

The costs reflect a considerable premium (approximately 80 percent) over current West Coast or Japanese fabrication of large jackets for waters offshore California. The premium is due to higher cost for the specified materials, welding procedures with a preheat requirements, a presumed rigorous inspection program, and the fabrication complexities, principally in the stiffened legs.

The piling are assigned slightly higher fabrication costs than the conductors because they would be fabricated in "can" sections, whereas the conductor could be fabricated as pipe in 40 feet lengths.

An allowance has been made for the yard improvements which include building ways, minor dredging, and minor bulkhead improvements.

Tables 6-6, 6-7 and 6-8 summarize the fabrication costs for the three base case designs.

6.3.4 Transportation

The transportation estimate includes all costs for mobilization, demobilization and operations of tugs and barges for transportation of all fabricated components to the offshore construction site.

The estimate has been based primarily on 9000 HP tugs due to the ready availability of this class of tug under U. S. flag. There are larger tug boats available worldwide; however, there may be "U. S. flag" restrictions on tows from a U. S. Port to the offshore construction sites. Also, it is felt that the total cost would not differ substantially if larger tugs were used.

The home base of all tugs and 400' x 100' cargo barges is assumed to be Puget Sound, while the launch barge is assumed to be mobilized from the North Sea. No demobilization is assumed for the launch barge.

The jacket and modules are assumed to be towed from the San Francisco Bay area or Japan, approximately 2500 miles to St. George Basin.

Table 6-9 shows transportation cost estimates for the three base case designs. It was felt that variations in transportation costs for the three basins would be minor; therefore, no distinction has been made.

6.3.5 Installation

The offshore installation costs include all derrick barge and other offshore operations related to installation of the jacket, piling conductors and placement (but not hookup) of deck modules.

Mobilization and Demobilization

Due to the severe weather conditions and the large total weight of modules to be lifted onto the jacket, a 1600/2000 T semi-submersible derrick barge has been assumed to be the centerpiece of the offshore construction spread. There are no vessels of this type in the Pacific Basin at this time, and the project team is aware of no firm plans to permanently station such a barge in the Pacific area. Therefore, we have included the cost of mobilizing a derrick barge from the North Sea. The mobilization costs are assumed to be composed of the following three components:

- The actual cost of towing the vessel.

- A reduced day rate taken to be 40 percent of the working rate, or $120,000/day.
- The insurance under tow taken to be 4 percent of the assumed replacement value of $150,000,000.

No demobilization costs are included, although certain unfavorable world-wide market conditions could dictate a duplication of the above-mentioned costs to return the vessel to its original base of operations. The practical limit in what a contractor may charge for mob/demob is limited by the owner's ability to have a new vessel built in Japan or Korea and partially amortized over the Bering Sea project.

The variability of mob/demob costs, the sensitivity of the topside module lift weights to crane limits, and the limited number of these vessels seems to suggest the commitment of a derrick barge very early in a project. This could be done by the award of a turnkey project or by negotiating mob/demob and a day rate with a contractor very early in the design phase. Cancellation penalties should be minor until the year or two prior to offshore work.

Operations

The operating spread consists of the following basic equipment:

- 1600/2000 T derrick barge
- Four offshore hammers, including two $1\overline{MM}$ to 1.5 $\overline{MM}$ ft-lb hammers.
- Diving spread
- Attendant tugs
- Grouting spread

- Supply vessel
- RCV spread
- Helicopter service
- Inspecting crews
- Painting crews
- Crew boats

The offshore day rate has been taken to be $300,000/day, based on the above spread with a supply base at Herendeen Bay.

The weather delays during the offshore construction season have been estimated to be approximately 30 percent for the period June through September, and increase very rapidly outside this window. This is based on the distribution of major storms as shown in Section 2 of this report and on operating history of this type of vessel in the North Sea during the summer season.

Table 6-10 summarizes the installation costs for the Bering Sea. It was felt that variations in installation costs for the three basins would be minor; therefore, no distinction has been made.

6.3.6 Summary of Base Cast Costs

Tables 6-11, 6-12, and 6-13 show the total estimated cost for the steel structures for St. George, Navarin, and North Aleutian Basins.

These summary tables include an allowance of 10 percent for home office costs, discussed in Section 10.1.5, and a 30 percent cost estimating allowance discussed in Section 10.1.4.

6.3.7 Sensitivity Cost Estimates

Tables 6-14 and 6-15 show the sensitivity of total steel structure cost to variations in water depth and production rate. All three basins are shown on each table. Less pronounced cost changes are evident for variations in environmental loading and soil conditions. Table 6-16 and Figure 6-3 indicate the variation in structure tonnage.

6.3.8 Cash Flow

Figures 6-4, 6-5 and 6-6 show estimated cash flows for St. George, Navarin and North Aleutian structures.

6.4 Concrete Structure

6.4.1 Items Included

This estimate contains all materials, equipment and labor costs associated with the following items:

- Fabrication of the concrete structure with all internal mechanical systems.
- Fabrication of the module support frame.
- Placement of modules on support frame in shipyard not including hookup between modules.
- Mating of integrated deck to GBS at a near shore site not including mechanical interface hookup.
- Towing and installation of complete structure to location.

6.4.2 Fabrication

The cost of the concrete substructure fabrication has been estimated on the basis of recent North Sea experience with concrete platforms and include allowances for extras which have occurred principally in the area of reinforcing steel. Other applicable data points relevant to fabrication in the Pacific Northwest are the Dome Tarsuit caissons, the Valdez floating container terminal, and major concrete floating bridge pontoons, all of which generally support the values used herein. From this background, the following concrete unit rates have been developed for the fabrication of concrete platforms of the size and complexity of those proposed for application in the Southern Bering Sea. A compilation of Norwegian sector platform costs has been made by Moe*, and some of this data has been used for gravity fabrication costs.

Concrete Substructures

	Storage	No Storage
Furnish and Place	$140/CY	140
Forms and Supports	$410/CY	410
Reinforcing	$230/CY	160
Prestressing	$180/CY	150
Total	$960/CY	$860/CY

Other principal items in the fabrication cost estimates which are in addition to the basic unit price include:

- Graving Dock - A large diameter, shallow draft construction basin in which the base raft of the caisson may be built in the dry.

- Steel Skirts and Dowels

- Launching and Mooring - Flooding the basin, float out and mooring of the base raft.

- Mooring and Fastenings - Supports for towing and mooring, pipeline risers, and internal decks and equipment.

- Heavy Weight Ballast - Iron ore or sand used to lower the center of gravity for stability during tow and installation.

*Cost Study, Norwegian Continental Shelf, "Recapitulation of the Development Evaluations and Recommendations," Report submitted to the Ministry of Petroleum and Energy by the Steering Group appointed by Royal Decree, Johannes Moe, Chairman, April 1980.

- Support Facilities for Construction Afloat - Barges, floating concrete plant, craneage, including tower cranes, crew boats, etc.

- Mechanical Systems - Ballast and drainage, oil transfer, underbase evacuation, underbase grouting, diesel oil.

- Electrical Systems - Instrumentation, power supply, oil measurement, cathodic protection.

- Utility Shaft Equipment and Decks - Includes elevators, ventilation, fire fighting.

- Riser and Drilling Shaft Equipment, Conductor Guides and Conductors.

- Module Support Frame - Structural frame, including transition connections to concrete shafts.

The fabrication cost estimates for the base case designs are summarized in Table 6-17 for St. George and Navarin and Table 6-18 for North Aleutian.

6.4.3 Mating, Towing and Installation

These operations encompass the towing of the substructure caisson to a deep water protected mating site, the transport of the deck to the same site, the deck mating operation, the inclining experiments, the towage to the site, followed by positioning, ballasting down to the seafloor, penetrating the skirts, and underbase grouting.

The mating, towing and installation cost estimates for the three base case designs are summarized in Table 6-19.

6.4.4 Summary of Base Case Costs

Tables 6-20 and 6-21 show the total estimated cost for the concrete structures for St. George/Navarin and North Aleutian Basins. These summary tables include an allowance of 10 percent for home office costs discussed in Section 6.1.5 and a 30 percent cost estimating allowance discussed in Section 6.1.4.

6.4.5 Sensitivity Cost Estimates

Tables 6-22, 6-23 and 6-24 show the sensitivity of total concrete structure cost to the following primary variables. All three basins are shown on each table.

- Water Depth - Table 6-22
- Storage Capacity - Table 6-23
- Production - Table 6-24

There are no postulated changes in cost due to soil conditions or environmental conditions; however, these variables limit the application of the chosen gravity structures as discussed in Section 5.

In addition, Tables 6-25 through 6-29 give breakdowns of fabrication costs for several of the sensitivity cases.

6.4.6 Cash Flow

Figures 6-7 and 6-8 show estimated cash flows for concrete structures in St. George/Navarin and North Aleutian.

6.5 Floating Production System

The following is a breakdown of the estimated costs for the floating production alternatives evaluated in this study. The following costs were based on the experience of the project team members plus input from the following organizations:

Sedco	Semi-submersibles
Santa Fe Drilling	Semi-submersibles
Interocean	Mooring quick release systems
Vetco	Subsea template and wellheads and risers
Imodco	Single point mooring
SBM	Single point mooring

6.5.1 Converted Drilling Semi--Ice Strength Modified

The converted drilling semi costs are based upon conversion of an existing SEDCO 700 series design semi-submersible.

- Cost of Existing, Non-converted Semi:	$100,000,000
(Cost is sensitive to market conditions at time of purchase.)	
- Cost of Conversion to Floating Production:	$ 15,000,000
(Cost based upon throughput of 25,000 BPD, flaring of gas and dumping of produced water)	
- Cost of Ice Reinforcement:	$ 10,000,000
(This retrofitting cost includes 600 T additional steel to be added. This gives the vessel capability to withstand 24-inch thick, 75 psi compressive strength ice failing on the vessel at both drilling (production) and transit draft.	
- Mooring Line Quick Release System	$ 1,000,000
(Based upon 12 units, 1.2 million lb. capacity ea.) Command unit and service module	
SUBTOTAL SEMI-SUBMERSIBLE	$126,000,000

- Subsea Production Systems and Riser

(The following costs are based upon Petrobras' Enchova Field production system, quoted from Vetco.

- Template		$ 3,000,000
(Cost is for six-well unitized template plus accommodation for four additional satellite wells.)		
- Production trees	$750,000 ea.x 6 wells	$ 4,500,000
(Use same cost for satellite trees)		
- Tubing hangers	$50,000 ea.x 6 wells	$ 300,000
- Manifold system		$ 1,500,000
- Production Riser (Assume 400' W.D.)		$ 2,500,000
- Pulling System Sales Riser		$ 400,000
- Wellheads (template $250,000 ea.x 6 wells)		$ 1,500,000
(Satellite wellheads $250,000)		
- Miscellaneous Tools		$ 300,000
- Allowance for Cold Temperature Modifications		$ 5,000,000
SUBTOTAL SUBSEA PRODUCTION		$19,000,000

- Single Point Mooring

  - Single point mooring cost : $ 40,000,000

    (This cost includes engineering, testing, fabrication and transportation. Cost of rigid yoke is included.)

  - 100,000 DWT storage tanker (surplus) $ 50,000,000

  - Storage vessel conversion cost $ 10,000,000

    (This cost includes modification for yoke connection plus ice strengthening of hull

  SUBTOTAL SPM $100,000,000

- Flowline and Subsea Installation Costs

    (These costs are basically an allowance for required derrick/lag barges for subsea installation, laying of flowlines and assistance in SPM installation)

  - Mobilization/Demobilization (800 T Derrick/Pipelay Barge) $ 5,000,000

  - Operating Costs

    (Assuming $200,000/day x 100 days) $ 20,000,000

  SUBTOTAL INSTALLATION $ 25,000,000

SUBTOTAL - INSTALLED SYSTEMS COST	$270,000,000
HOME OFFICE COST + 10%	$ 27,000,000
CONTINGENCY @ 30%	$ 89,000,000
GRAND TOTAL	$386,000,000

6.5.2 Converted Drilling Semi - Nonmodified

The cost for this production alternative is the same as the preceding alternative, less the $10 million retrofitting costs for ice strengthening.

6.5.3 Purpose-Built Ice Class Semi-Submersible

- SEDCO 900 Series Ice Class Semi	$200,000,000
Deckload capacity would be about 4500 T at drilling (production) draft.)	
or Santa Fe Sub-Arctic Semi	
The vessel has a variable deck capacity of about 6000 T.)	
The following costs are assumed to be the same as for the converted semi:	
- Subsea production system and riser	$ 19,000,000
- Single Point Mooring/Storage (including tanker modification)	$100,000,000
- Flowline and subsea installation	$ 25,000,000
SUBTOTAL - INSTALLED SYSTEM COST	$344,000,000
HOME OFFICE @ 10%	$ 34,000,000
CONTINGENCY @ 30%	$113,000,000
GRAND TOTAL	$491,000,000

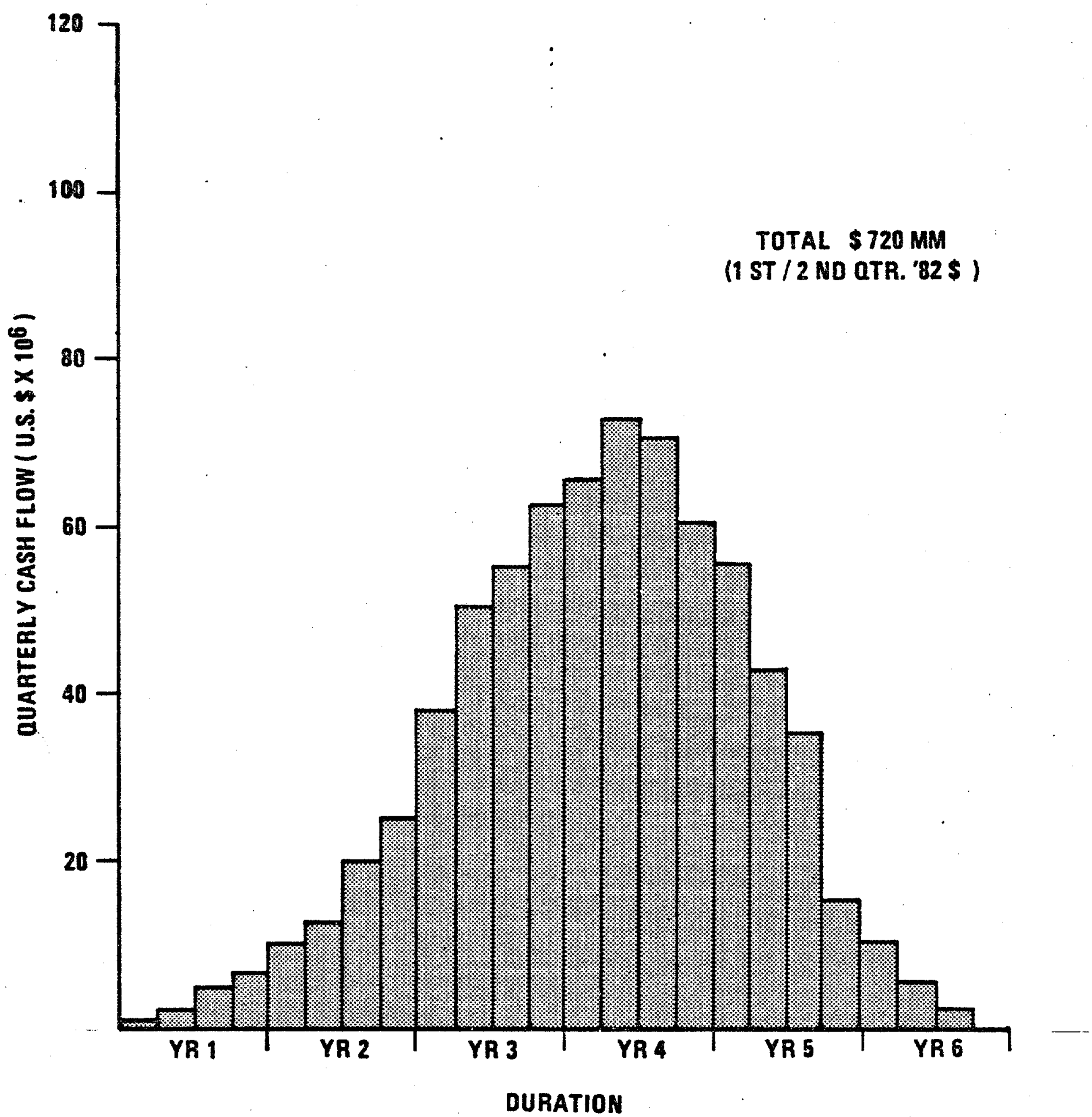
6.6 Schedules

Schedules for platforms are shown on the following figures:

Steel Platform and Topside	- Figure 6-9
Concrete Platform and Topside	- Figure 6-10
Floating Production System (Purpose Built Semi)	- Figure 6-11

There will be minor variations in these schedules for the different basins and for differnt sensitivity cases. These changes are only a matter of a few months at most and are not delineated.

The schedule assumes a fast track basis where prime fabrication contracts are awarded when 70 to 80 percent of the engineering is complete. At this point, the vast majority of major cost generating items have been defined, and there should be an insignificant cost penalty for this fast track contracting plan.



* EXCLUDES: MODULE SUPPORT FRAME, SCOPE / COSTS BELOW MODULE SUPPORT FRAME, INSTALLATION BARGE COSTS, & SITE SHUTTLE COSTS.

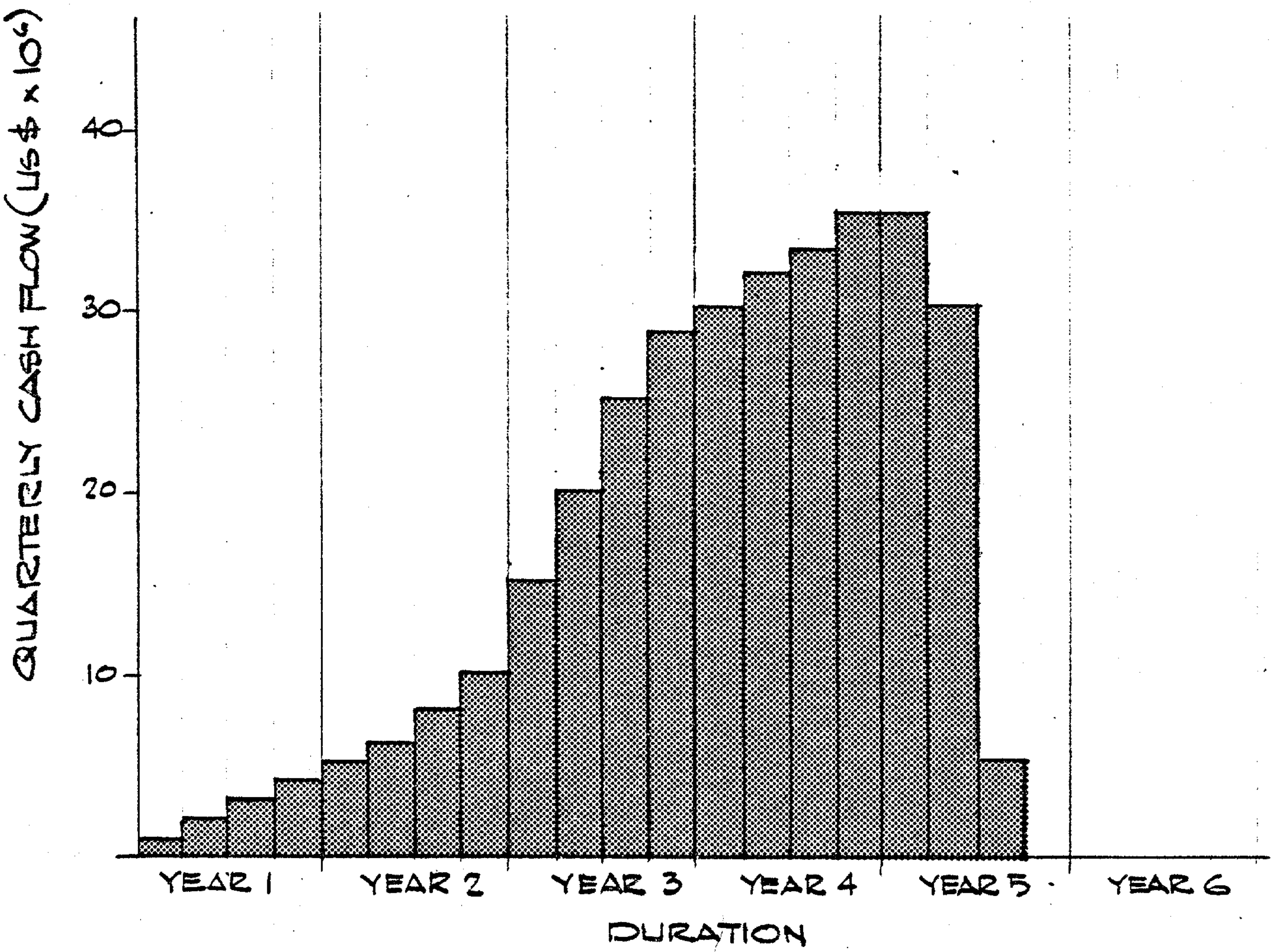
Figure 6 – 2

Southern Bering Sea Production Systems Study

**ESTIMATED CASH FLOW
TOPSIDE FACILITIES *
FOR STEEL STRUCTURE
(300 MBPD)**

Job 15154 BECHTEL June 1982

TOTAL: $327 MILLION



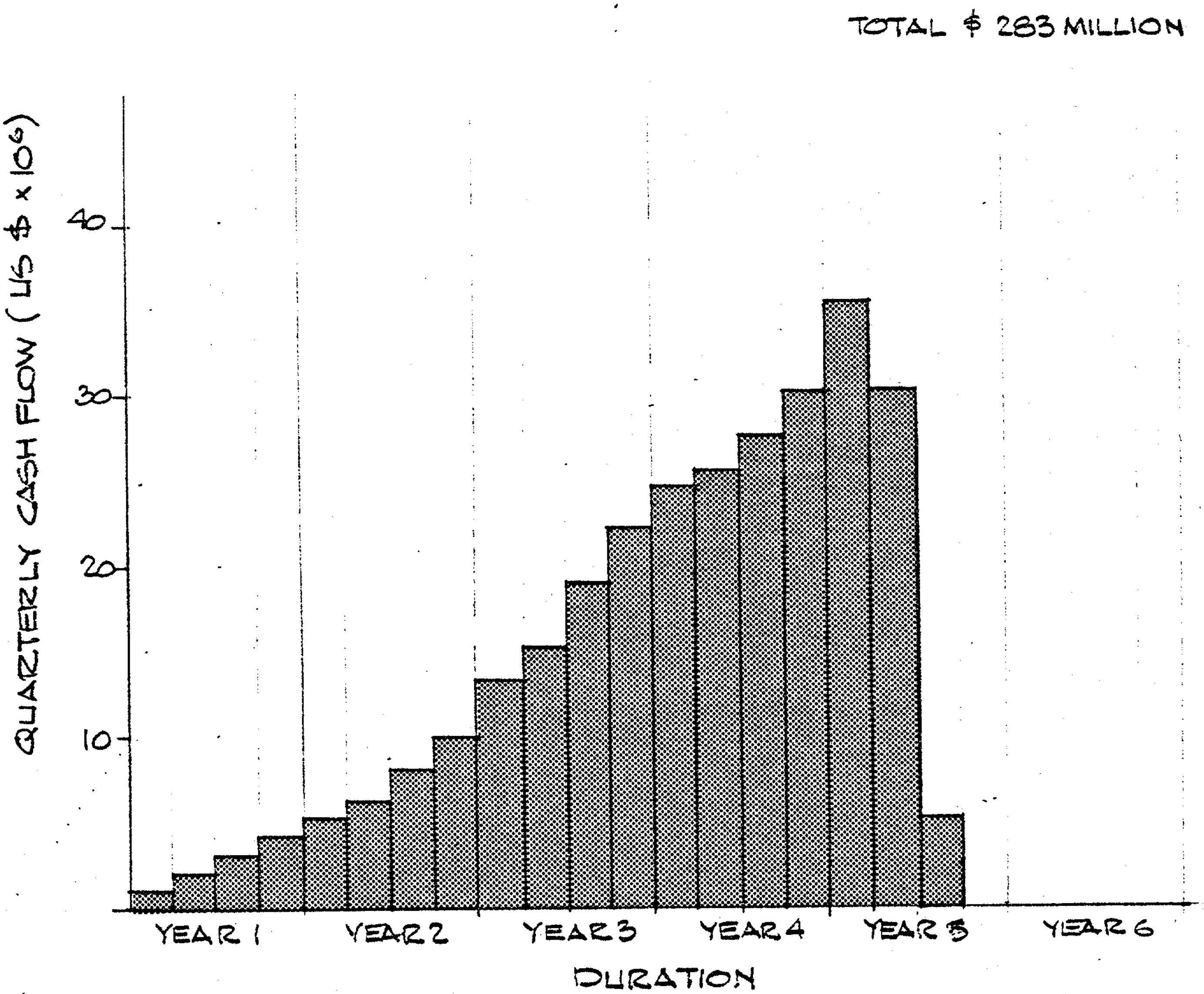
* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUD

ST. GEORGE STEEL STRUCTURE *

ESTIMATED CASH FLOW

FIGURE 6-4



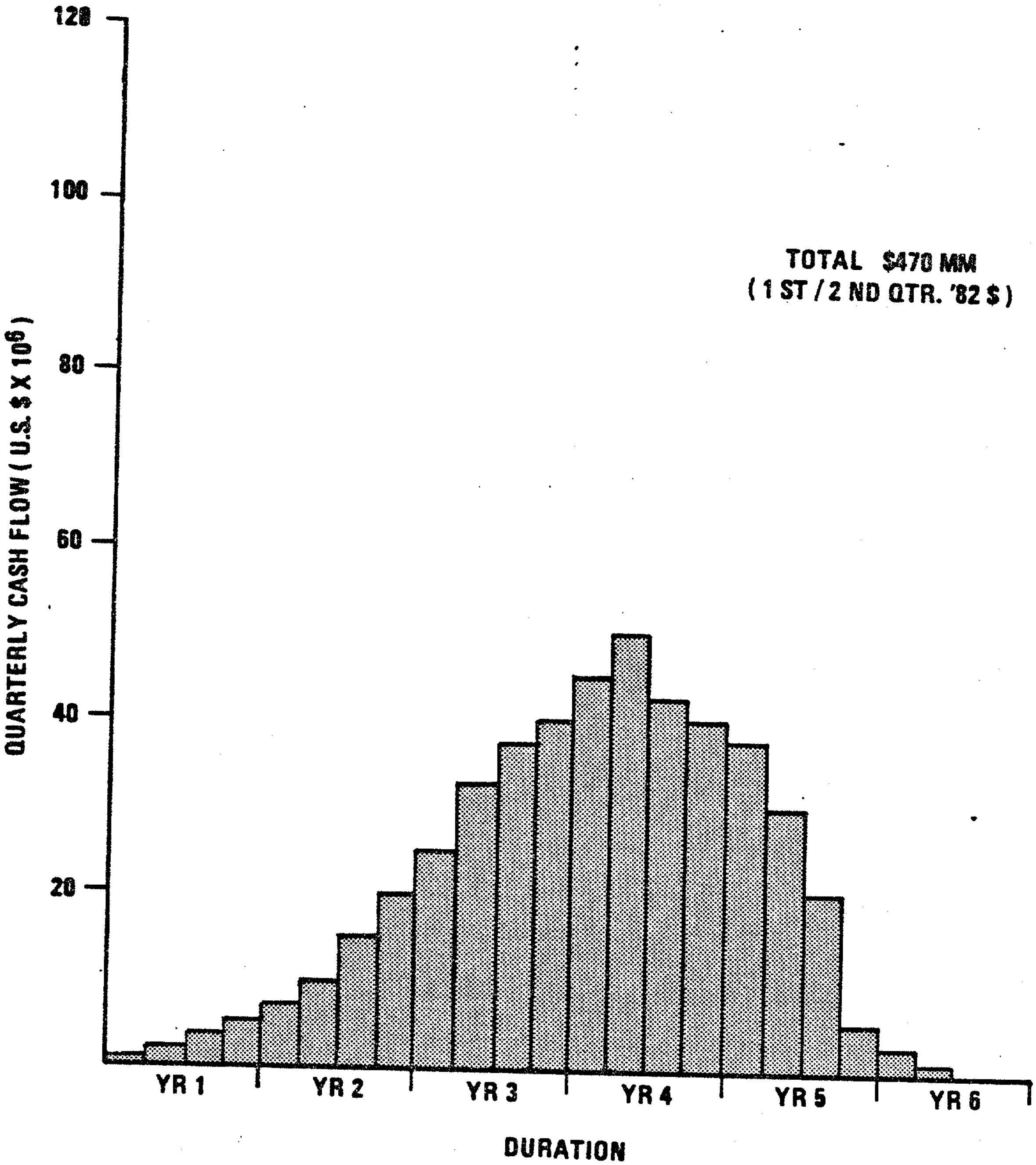
* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUD

NORTH ALEUTIAN STEEL STRUCTURE *

ESTIMATED CASH FLOW

FIGURE 6-6



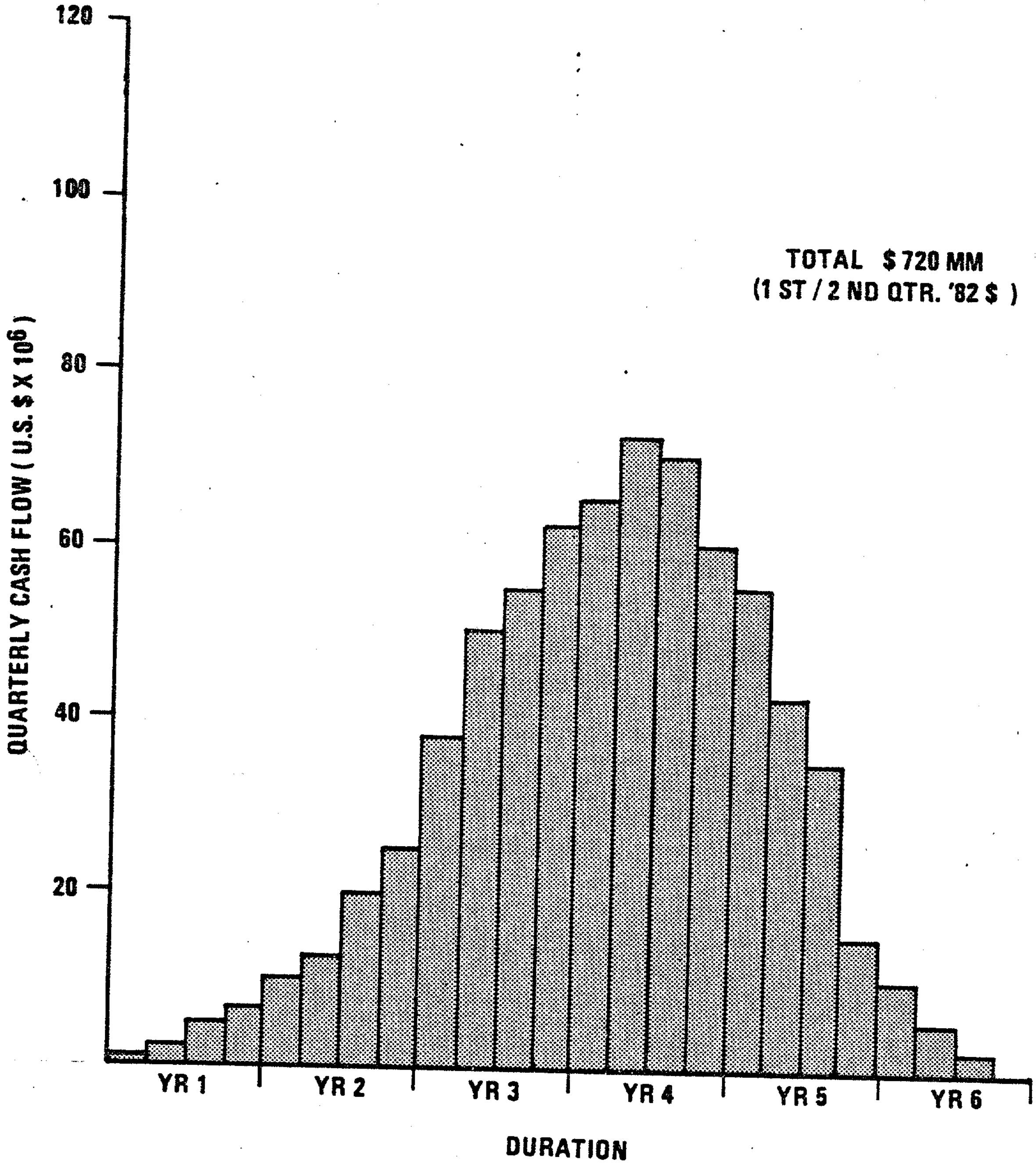
* EXCLUDES: MODULE SUPPORT FRAME, SCOPE / COSTS BELOW MODULE SUPPORT FRAME, INSTALLATION BARGE COSTS, & SITE SHUTTLE COSTS.

Figure 6 – 1

Southern Bering Sea Production Systems Study

ESTIMATED CASH FLOW
TOPSIDE FACILITIES *
FOR STEEL STRUCTURE
(100 MBPD)

Job 15154 BECHTEL June 1982



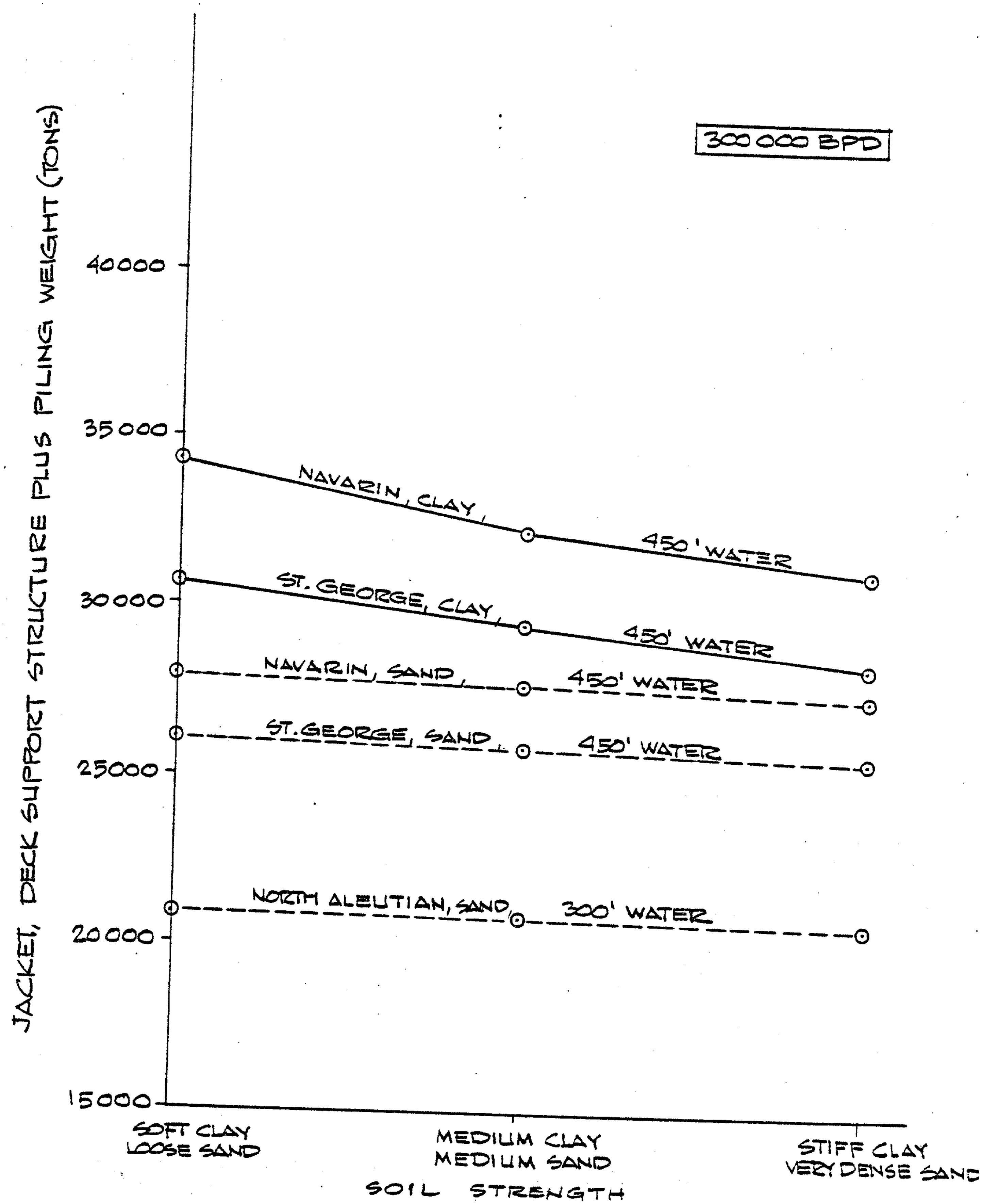
* EXCLUDES: MODULE SUPPORT FRAME, SCOPE / COSTS BELOW MODULE SUPPORT FRAME, INSTALLATION BARGE COSTS, & SITE SHUTTLE COSTS.

Figure 6 – 2

Southern Bering Sea Production Systems Study

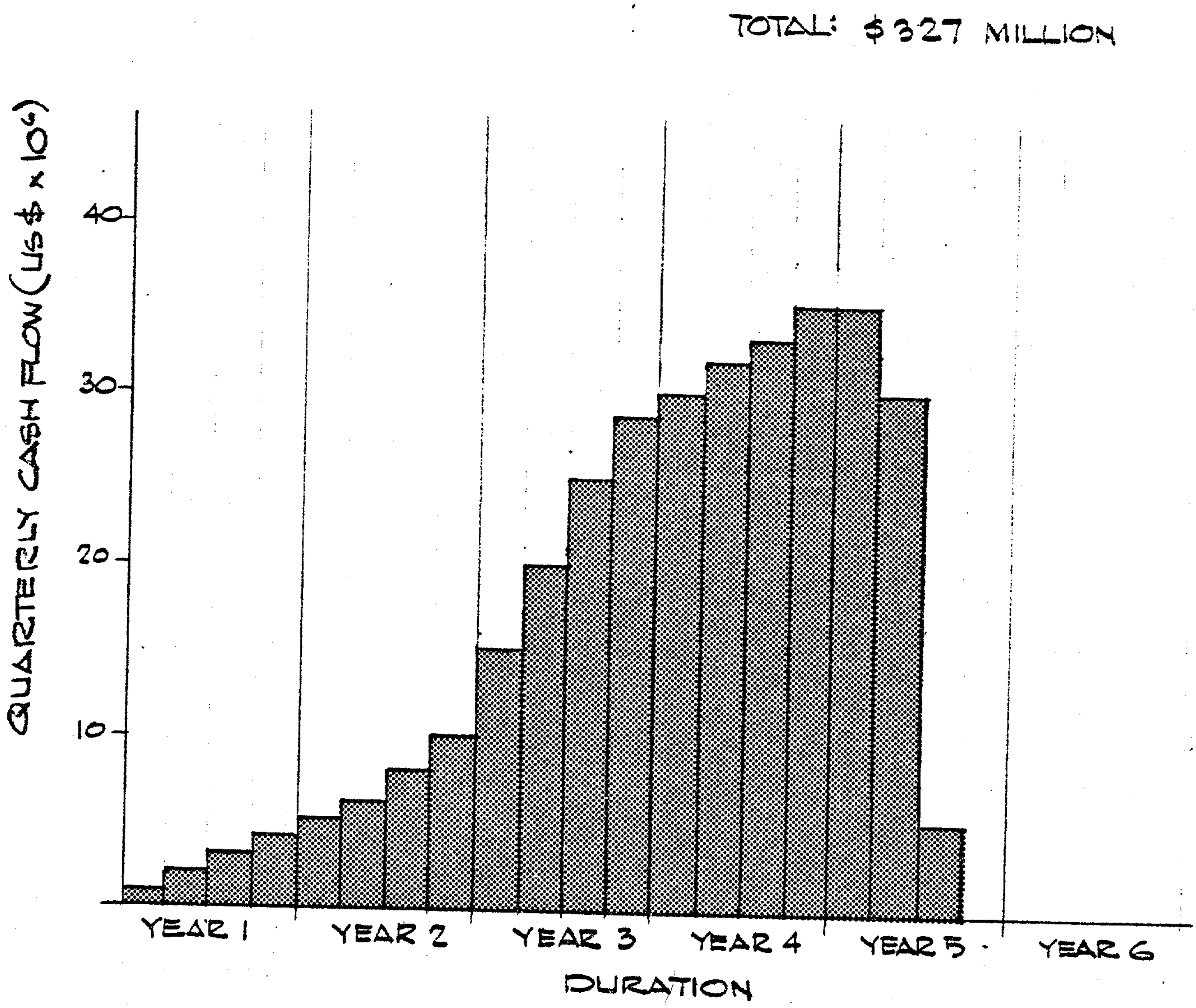
ESTIMATED CASH FLOW TOPSIDE FACILITIES * FOR STEEL STRUCTURE (300 MBPD)

Job 15154 BECHTEL June 1982



VARIATION OF STRUCTURE WEIGHT WITH SOIL STRENGTH

FIGURE 6-3



* EXCLUDES TOPSIDES & HOOKUP

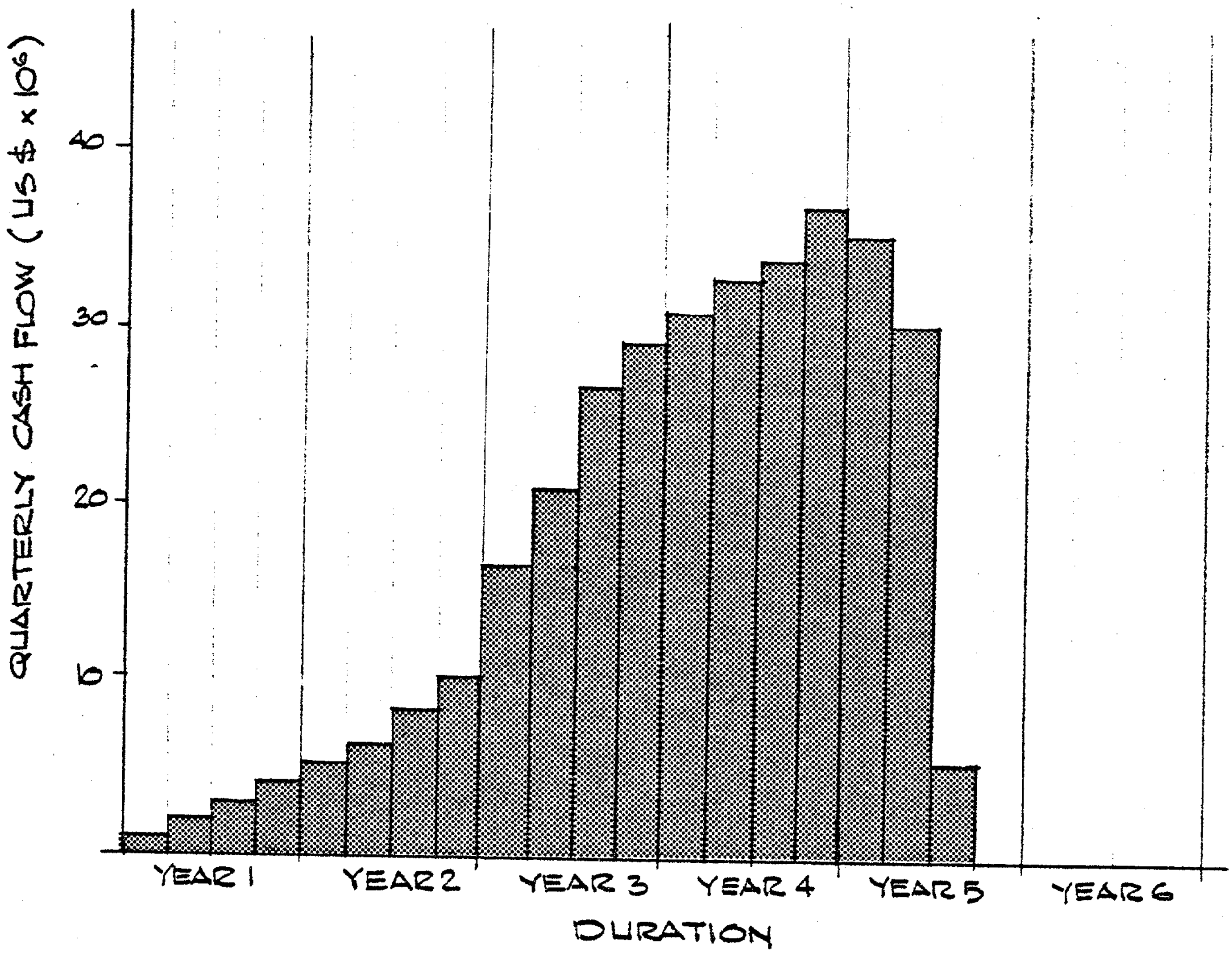
SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

ST. GEORGE STEEL STRUCTURE *

ESTIMATED CASH FLOW

FIGURE 6-4

TOTAL: $338 MILLION



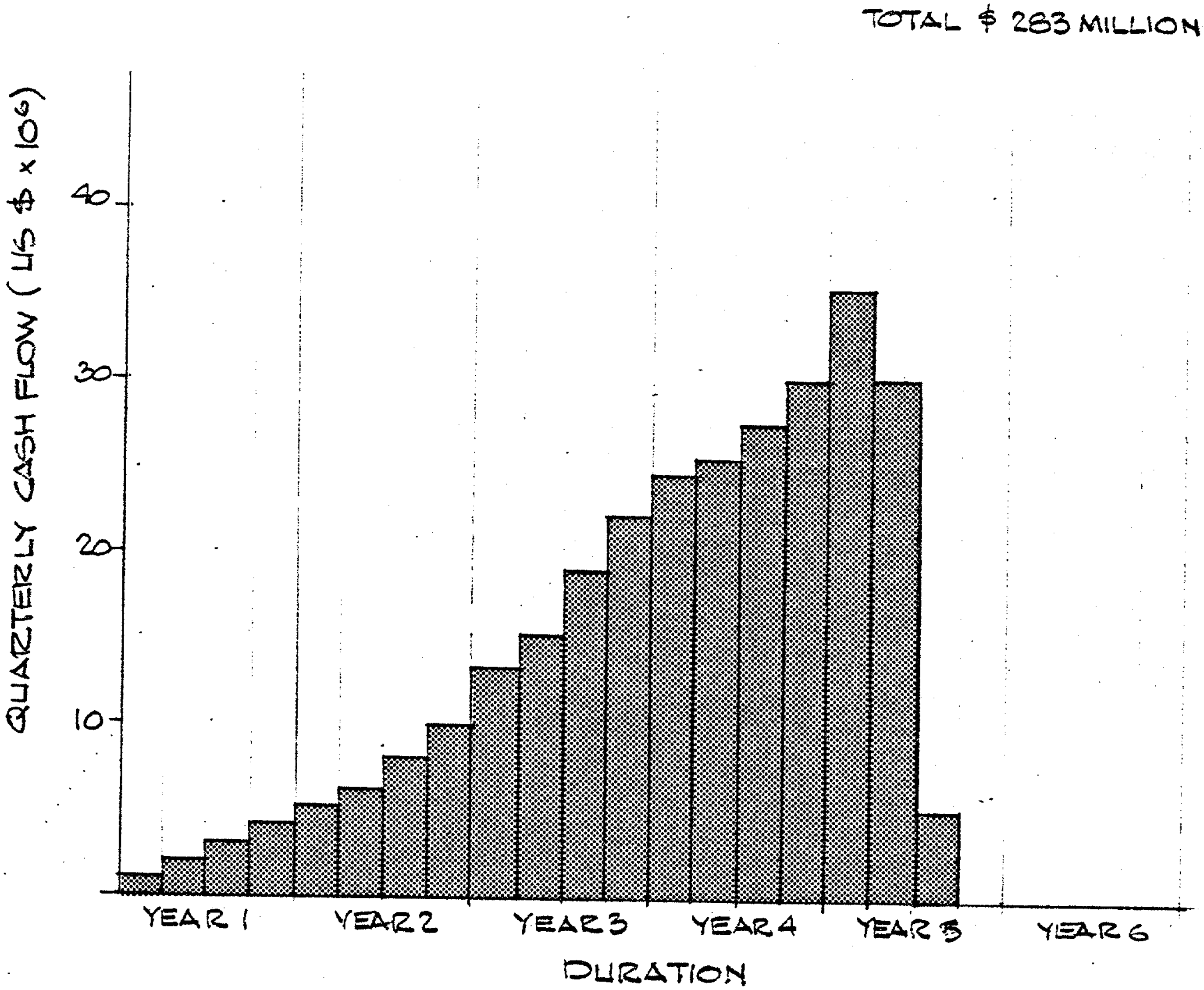
* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

NAVARIN STEEL STRUCTURE *

ESTIMATED CASH FLOW

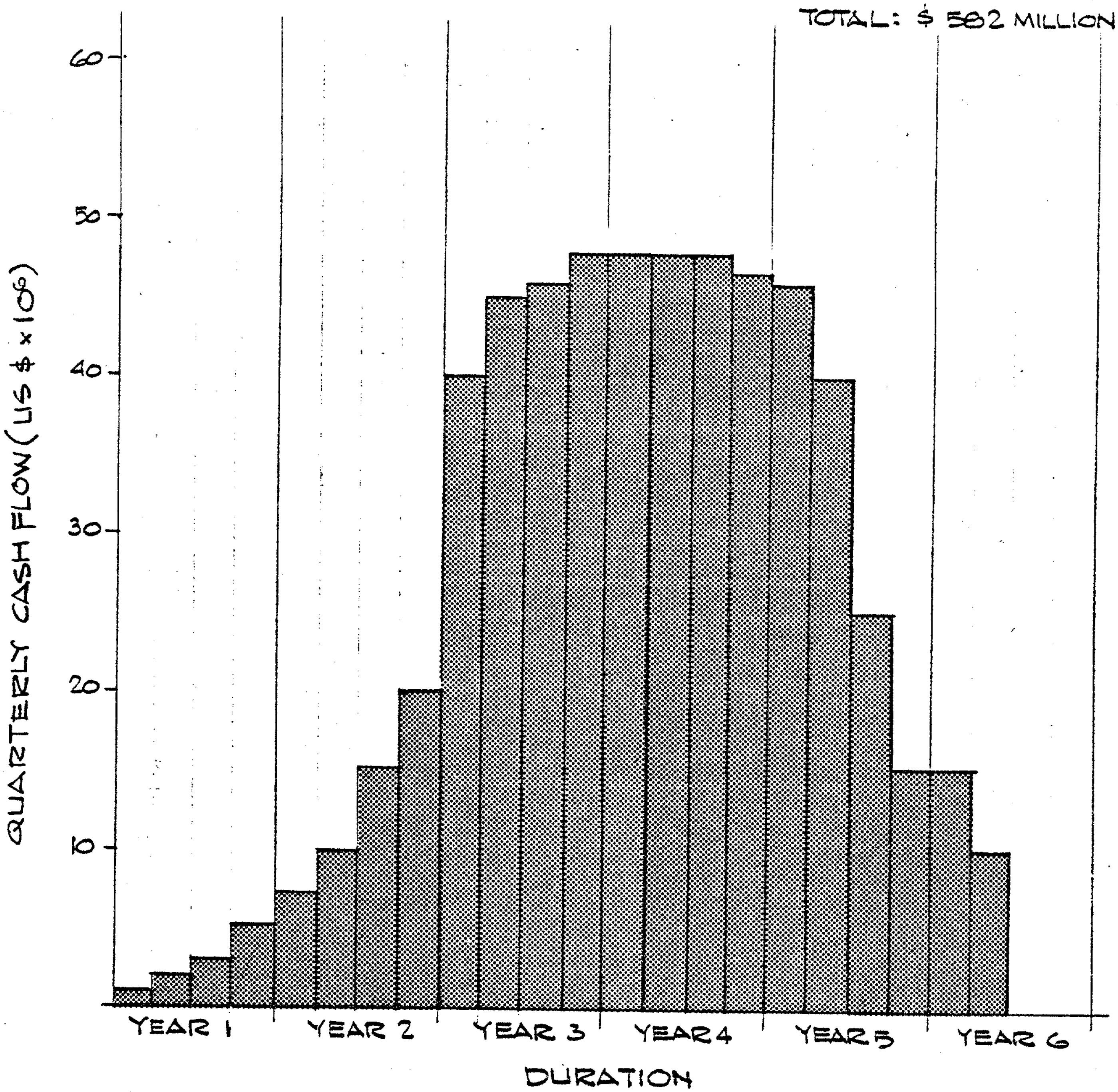
FIGURE 6-5



* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
NORTH ALEUTIAN STEEL STRUCTURE *
ESTIMATED CASH FLOW

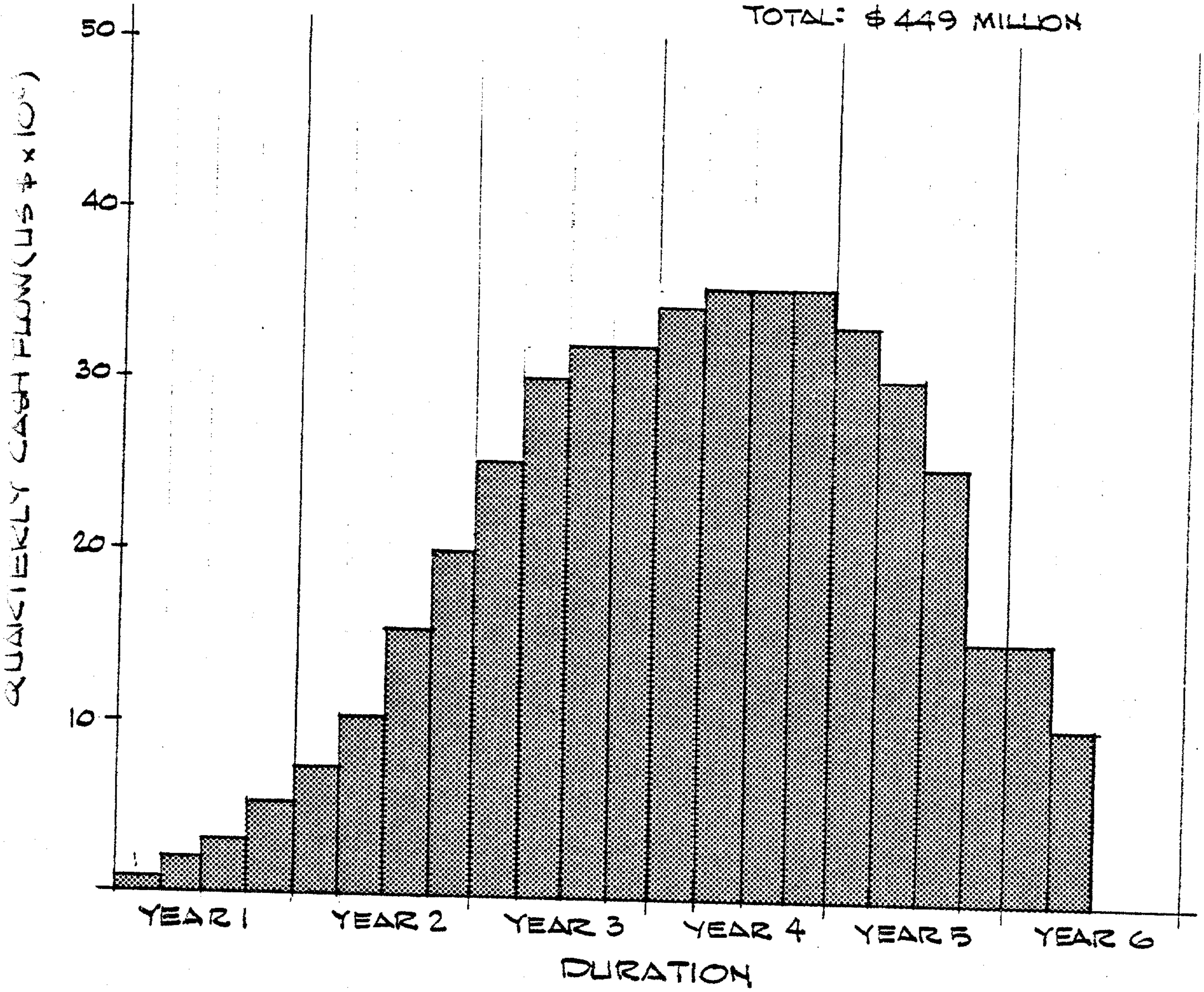
FIGURE 6-6



* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
ST. GEORGE OR NAVARIN CONCRETE STRUCTURE*
ESTIMATED CASH FLOW

FIGURE 6-7



* EXCLUDES TOPSIDES & HOOKUP

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY
NORTH ALEUTIAN CONCRETE STRUCTURE *
ESTIMATED CASH FLOW

FIGURE 6-8

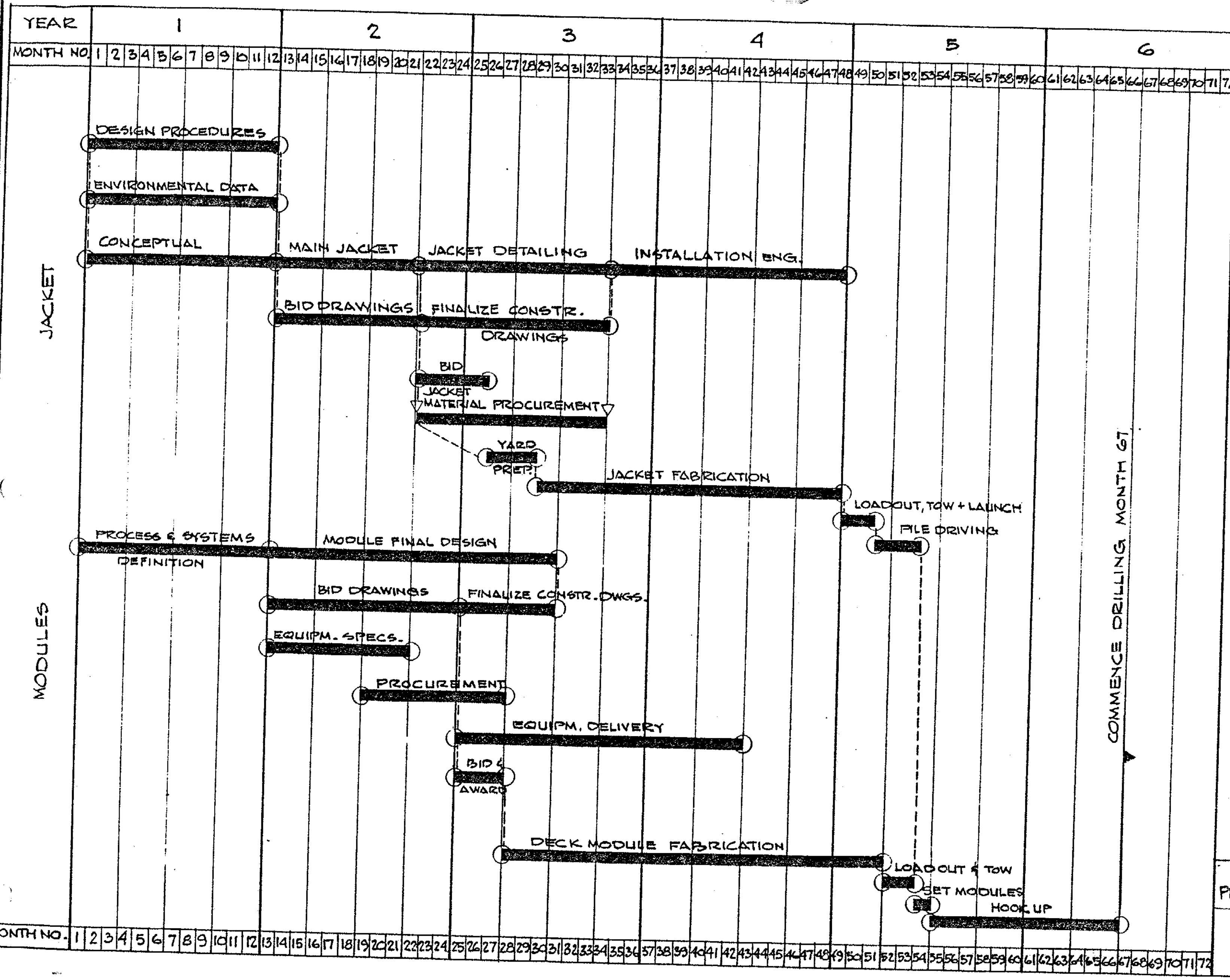


FIGURE 6-9

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

MASTER SCHEDULE STEEL PLATFORM

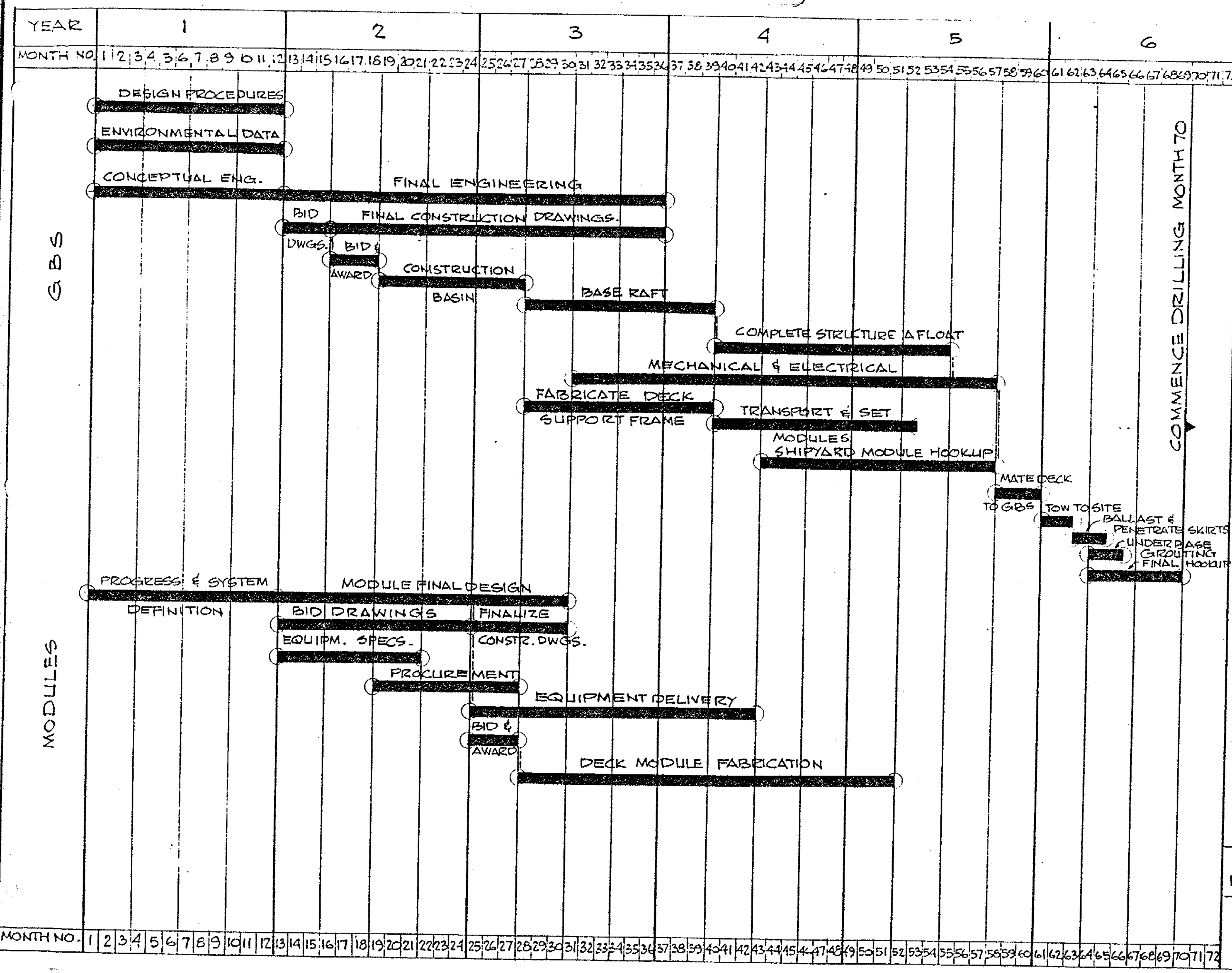
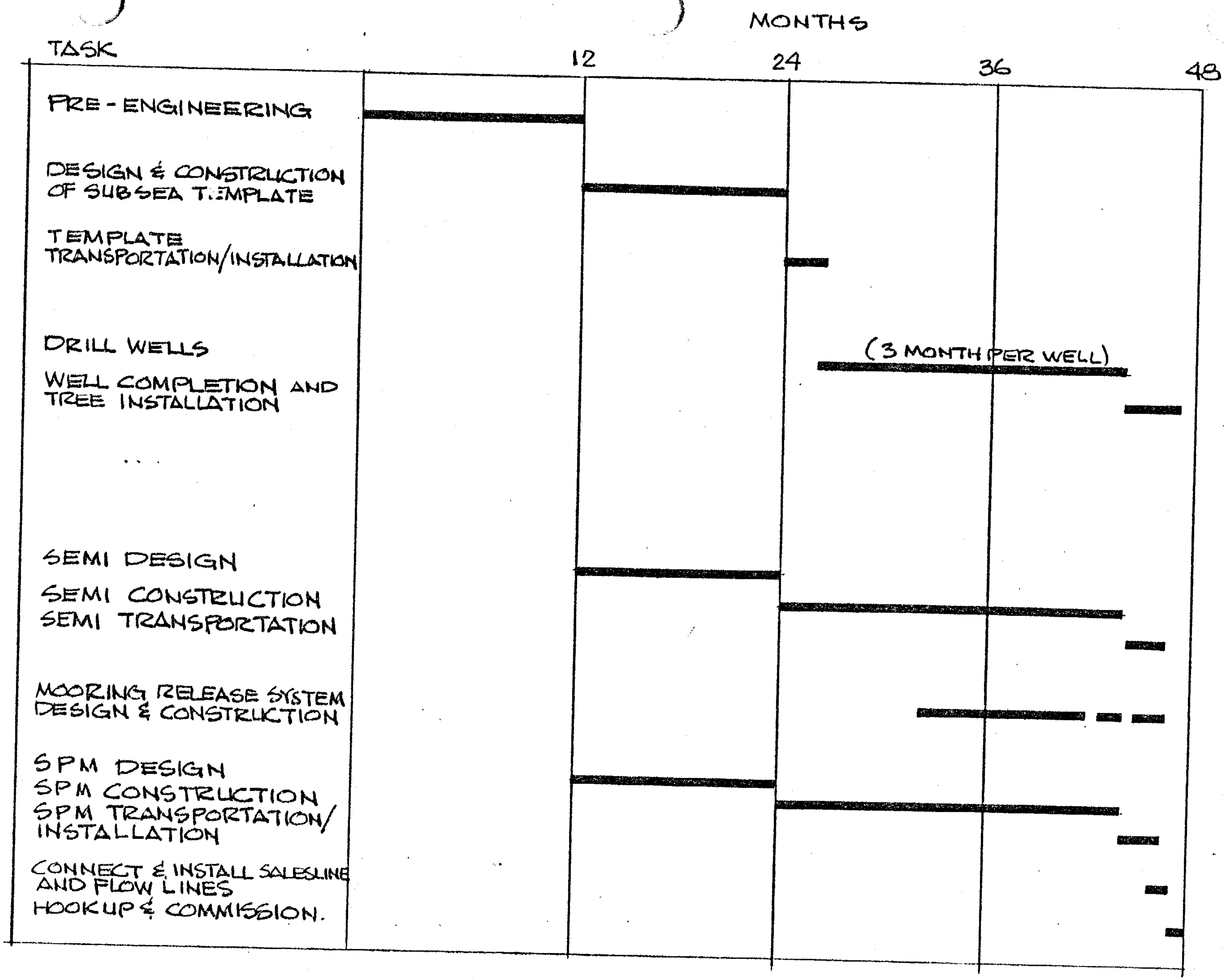


FIGURE 6-10

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

MASTER SCHEDULE
CONCRETE PLATFORM



PROJECT SCHEDULE - PURPOSE BUILT SEMI

Table 6 - 1

TOPSIDE FACILITIES MATERIAL AND FABRICATION COSTS

(Costs in U.S.Dollars x 10^3)

Modules	Throughputs 100 MBPD	300 MBPD
Living quarters + Helideck Installation	360 beds @ 78/bed + 3,000 = 31,000	432 beds @ 86/bed + 3,000 = 40,000
Drilling	2 rigs @ 17,500 ea. = 35,000	2 rigs @ 20,500 ea. = 41,000
Production	100,000 BPD @ 0.410/BPD = 41,000	300,000 BPD @ 0.340/BPD = 102,000
Power Generation	30 MW @ 860/MW = 60,000	100 MW @ 810/MW = 81,000
Utilities	360 men @ 64/man = 23,000	432 men @ 90/man = 39,000
Flareboom	100,000 BPD @ $0.020/BPD = 2,000	300,000 BPD @ 0.033/BPD = 7,000
Other Equipment (allowance)	15,000	18,000
TOTAL	207,000	328,000

Note: Costs for the 100 MBPD case and 300 MBPD case were taken from different data bases as outlined in Section 6.2.2.1; therefore, there are different unit costs assigned to the various items above for the two production cases.

Table 6 - 2

TOPSIDE HOOKUP COSTS

Activity	Throughputs 100 MBPD	Throughputs 300 MBPD
Steel Platform		
Module skidding, tiedown and life support hookup (allowance)	$\$6x10^6$	$\$7x10^6$
Offshore module hookup (12 mos.)	$1.3x10^6$ mh @ \$85/mh = $\$111x10^6$	$2x10^6$ mh @ \$85/mh = $\$170x10^6$
	$\$117x10^6$	$\$177x10^6$
Concrete Platform (with utility shaft)		
Nearshore (shipyard) module placement, hookup and sea fastening	$1.0x10^6$ mh @ \$55/mh = $\$55x10^6$	$1.5x10^6$mh @ \$55/mh = $\$83x10^6$
Nearshore (remote, deep draft area) topside/utility shaft interfacing	$0.35x10^6$ mh @ \$140/mh = $\$49x10^6$	$0.5x10^6$ mh @ \$140/mh = $\$70x10^6$
Offshore final hookup (6 mos.)	$0.6x10^6$ mh @ \$85/mh = $\$51x10^6$	$0.65x10^6$ mh @ \$85/mh = $\$55x10^6$
	$\$155x10^6$	$\$208x10^6$

Note: Transportation support costs shown separately on Table 6 - 3.

Table 6 - 3

TRANSPORTATION COSTS DURING HOOKUP (1), (8)
(U.S. Dollars x 10^6)

Steel Platform - (12 months offshore hookup period)

	North Aleutian	St. George	Navarin
Air Transportation (2)	4	5	16
Supply Boats	11 (3)	18 (4)	35 (5)
Safety Vessel	6 (6)	6 (6)	9 (7)
	21	29	60

Concrete Platform - (6 months offshore hookup period)

	North Aleutian	St. George	Navarin
Air Transportation (2)	2	2.5	8
Supply Boats	5.5 (3)	9 (4)	17.5 (5)
Safety Vessel	3 (6)	3 (6)	4.5 (7)
	10.5	14.5	30

Notes -

(1) No contingency or home office included.
(2) Order of magnitude quotations.
(3) 3 @ $10,000/day ea.
(4) 5 @ $10,000/day ea.
(5) 8 @ $12,000/day ea.
(6) 1 @ $15,000/day
(7) 1 @ $25,000/day
(8) No difference is taken between the two production cases. There are about 20% more personnel at the peak periods of the 300,000 BPD case, but it is felt the transportation spreads would be roughly the same.

Table 6 - 4

TOTAL TOPSIDE FACILITIES COSTS
STEEL STRUCTURE
(U.S. Dollars x 10^6)

	Throughputs 100 MBPD	300 MBPD
Module material & fabrication	207	328
Hookup	117	177
Offshore Transportation		
o North Aleutian	21	21
o St. George	29	29
o Navarin	60	60
Subtotal		
o North Aleutian	345	526
o St. George	353	534
o Navarin	384	565
Home Office and Contingency*		
o North Aleutian	148	226
o St. George	152	230
o Navarin	165	243
Total		
o North Aleutian	493	752
o St. George	505	764
o Navarin	549	808

* Home office taken as 10% and contingency as 30% of total or 1.43 total factor.

Table 6 - 5

TOTAL TOPSIDE FACILITIES COSTS
CONCRETE STRUCTURE WITH BASE STORAGE
(U.S. Dollars x 10^6)

	Throughputs 100 MBPD	Throughputs 300 MBPD
Module Material and Fabrication	207	328
Hookup	155	208
Offshore Transportation		
o North Aleutian	10.5	10.5
o St. George	14.5	14.5
o Navarin	30	30
Subtotal		
o North Aleutian	373	547
o St. George	377	551
o Navarin	392	566
Home Office and Contingency*		
o North Aleutian	160	235
o St. George	162	237
o Navarin	169	243
Total		
o North Aleutian	533	782
o St. George	539	788
o Navarin	561	809

*Home Office taken as 10% and contingency as 30% of total or 1.43 total factor.

TABLE 6-6

450' W.D. BERING SEA STEEL STRUCTURE

FOR ST. GEORGE BASIN - 300,000 BPD

FABRICATION COST ESTIMATE

1.	Jacket, Deck Substructure and Appurtenances[1] 20380 T @ 5,500/T	$112,090,000
2.	Piles 9030 T @ 1,800/T	16,254,000
3.	Conductors (48)[2] 4800 T @ 1600/T	7,680,000
4.	Onshore Skids and Yard Improvements[3]	5,000,000
	Total	$141,024,000

[1]Assume principally A537 or A633 steels are used.

[2]Assumes approximately 300' penetration into seafloor.

[3]Includes dredging, bulkhead, building ways.

TABLE 6-7

450' W.D. BERING SEA STEEL STRUCTURE

FOR NAVARIN BASIN - 300,000 BPD

FABRICATION COST ESTIMATE

1.	Jacket, Deck Substructure and Appurtenances[1]	$115,390,000
	20980 T @ 5,500/T	
2.	Piles	20,286,000
	11270 T @ 1,800/T	
3.	Conductors (48)[2]	7,680,000
	4800 T @ 1600/T	
4.	Onshore Skids and Yard Improvements[3]	5,000,000
	Total	$148,356,000

[1]Assume principally A537 or A633 steels are used.

[2]Assumes approximately 300' penetration into seafloor.

[3]Includes dredging, bulkhead, building ways.

TABLE 6-8

300' W.D. BERING SEA STEEL STRUCTURE

FOR NORTH ALEUTIAN - 300,000 BPD

FABRICATION COST ESTIMATE

1.	Jacket, Deck Substructure and Appurtenances[1]	$90,640,000
	16480 T @ 5,500/T	
2.	Piles	7,830,000
	4350 T @ 1,800/T	
3.	Conductors (48)[2]	6,320,000
	3950 T @ 1600/T	
4.	Onshore Skids and Yard Improvements[3]	5,000,000
	Total	$109,790,000

[1]Assume principally A537 or A633 steels are used.

[2]Assumes approximately 300' penetration into seafloor.

[3]Includes dredging, bulkhead, building ways.

TABLE
TRANSPORTATIO STS FOR
BERING SEA STEEL STRUCTURES

	Mobilization	Transport	Demobilization	Standby*	Total
1. Eight Module Cargo Barges (400' x 100' with 1-9000 HP Tug Each)					
a. Route	P.S. to S.F.	S.F. to B.S.	B.S. to P.S.		
b. Assumed Mileage	750	2,500	1,800		
c. Days (each)	4	12	8	30	
d. Daily Rate (each)	$20,500	$20,500	$20,500	$13,500	
e. Total	$ 656,000	$1,968,000	$1,312,000	$3,240,000	$7,176,000
2. Two Piling and Conductor Barges (400'x100' with 1-9000 HP Tug Each)					
a. Route	P.S. to S.F.	S.F. to B.S.	B.S. to P.S.		
b. Assumed Mileage	750	2,500	1,800		
c. Days (each)	4	12	8	30	
d. Daily Rate (each)	$20,500	$20,500	$20,500	$13,500	
e. Total	$ 164,000	$ 492,000	$ 328,000	$ 810,000	$1,794,000
3. 600' Launch Barge (1-9000 HP Tug Unloaded) (2-9000 HP Tugs Loaded)					
a. Route	N.S. to S.F.	S.F. to B.S.			
b. Assumed Mileage	12,500	2,500			
c. Days (each)	180	36		70	
d. Daily Rate (each)	$46,000	$62,000		$13,500	
e. Total	$8,280,000	$2,232,000		$2,800,000	$13,312,000
4. TOTALS	$9,100,000	$4,692,000	$1,640,000	$6,850,000	$22,282,000

Notes:

* Includes weather
P.S. Puget Sound
S.F. San Francisco
B.S. Bering Sea (St. George)
N.S. North Sea

TABLE 6-10
450' W.D. BERING SEA STEEL STRUCTURE
INSTALLATION COST ESTIMATE

1.	Derrick Barge Mobilization[1,2]		
	- Towing Costs 15,000 miles, 120 days @ $16,000 (From North Sea)		$ 1,920,000
	- Rental Under Tow - 120 Days @ $120,000		14,400,000
	- Insurance Under Tow		6,000,000
	- Weather @ 30%		4,600,000
2.	Derrick Barge Operations - 120 Days @ $300,000		36,000,000
	- Run Anchors and Setup	2 Days	
	- Position and Ballast Downjacket	1 Day	
	- Drive 18 Piles	34 Days	
	- Drive 48 Conductors	24 Days	
	- Grouting and Welding	6 Days	
	- Install Modules - 20 Each	25 Days	
	- Weather Delays @ 30%	28 Days	
3.	Support Boats and Helicopter - 120 Days @ $40,000		$ 4,800,000
	TOTAL		$67,720,000

Notes

[1]Assumes a 2000 ton semi-submersible barge

[2]Assumes only one-way mobilization

TABLE 6-11

450' W.D. BERING SEA STEEL STRUCTURE

FOR ST. GEORGE BASIN - 300,000 BPD

COST SUMMARY

(TO NEAREST MILLION)

Fabrication	$141,000,000
Transportation	22,000,000
Installation	68,000,000
Home Office @ 10%[1]	23,000,000
Contingency @ 30%	76,000,000
Total	$330,000,000

[1]Includes all engineering, procurement, and project management

TABLE 6-12

450' W.D. BERING SEA STEEL STRUCTURE

FOR NAVARIN BASIN - 300,000 BPD

COST SUMMARY

(TO NEAREST MILLION)

Fabrication	$148,000,000
Transportation	22,000,000
Installation	68,000,000
Home Office @ 10%[1]	24,800,000
Contingency @ 30%	79,000,000
Total	$341,000,000

[1]Includes all engineering, procurement, and project management

TABLE 6-13

300' W.D. BERING SEA STEEL STRUCTURE

FOR NORTH ALEUTIAN SHELF - 300,000 BPD

COST SUMMARY

(TO NEAREST MILLION)

Fabrication	$110,000,000
Transportation	22,000,000
Installation	68,000,000
Home Office @ 10%[1]	20,000,000
Contingency @ 30%	66,000,000
Total	$286,000,000

[1]Includes all engineering, procurement, and project management

TABLE 6-14

BERING SEA STEEL STRUCTURE COST (1982 U.S. $\$ \times 10^6$)

FOR VARIATIONS IN WATER DEPTH (300000 BPD)

	St. George Basin			Navarin Basin			N. Aleutian Shelf		
Water Depth	300'	450'	600'	300'	450'	600'	200'	300'	350'
Fabrication	116.9	141.	176.6	122.	148.4	194.7	83.7	109.8	120.1
Transportation	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3
Installation	67.7	67.7	67.7	67.7	67.7	67.7	67.7	67.7	67.7
Home Office @ 10%	20.7	23.1	26.7	21.2	23.8	28.5	17.4	20.0	21.0
Contingency @ 30%	68.3	76.2	88.0	70.0	78.7	95.0	57.3	68.0	69.3
TOTAL	292.9	327.3	378.3	300.2	337.9	404.2	245.4	282.8	297.4

TABLE 6-15

BERING SEA STEEL STRUCTURE COST (1982 U.S. $\$x10^6$)

FOR 100000 BPD and 300000 BPD PRODUCTION

	450' Water St. George Basin		450' Water Navarin Basin		300' Water N. Aleutian Shelf	
Production	100000	300000	100000	300000	100000	300000
Fabrication	123.8	141.	131.6	148.4	99.0	109.8
Transportation	22.3	22.3	22.3	22.3	22.3	22.3
Installation	67.7	67.7	67.7	67.7	67.7	67.7
Home Office @ 10%	214	23.1	22.2	23.8	18.9	20.0
Contingency @ 30%	70.6	78.2	73.2	78.7	62.4	66.0
TOTAL	302.8	327.3	314.0	337.9	267.3	282.8

TABLE 6-16

TONNAGE OF JACKET, DECK SUPPORT STRUCTURE
AND PILING FOR VARIATION IN ENVIRONMENTAL LOADS

	Lower Bound	Design Basis	Upper Bound
St. George	Small decrease (storm, H = 75')	29410 (storm, H = 85')	Small increase (storm, H = 90°)
Navarin	31240 (ice, 80 kips/ft)	32250 (ice, 115 kips/ft)	37980 (ice, 140 kips/ft)
N. Aleutian	Small decrease (storm, H = 65')	20830 (storm, H = 78' ice, 85 kips/ft)	Small increase (ice, 110 kips/ft)

1. Production Rate 300000 BPD for all basins.

2. Water Depth = 450 feet for St. George and Navarin
   = 300 feet for North Aleutian

3. Parentheses indicate the dominant environmental loading.

TABLE 6-17

CONCRETE GRAVITY STRUCTURE

ST. GEORGE OR NAVARIN - BASE CASE

(450' WATER DEPTH, 1,200,000 BARREL STORAGE)

FABRICATION COST ESTIMATE

1.	Graving Dock	$ 10,000,000
	Land Acquisition	
	Graving Dock Construction	
2.	Concrete (228,000 CY @ $960)	218,880,000
3.	Steel Skirts and Dowels	4,000,000
4.	Launching and Mooring	2,800,000
5.	Inserts and Fastenings	1,900,000
6.	Heavy Weight Ballast	2,600,000
7.	Support Facilities for Construction Afloat	4,700,000
8.	Mechanical Systems	44,000,000
9.	Electrical Systems	4,700,000
10.	Utility Shaft Equipment and Decks	27,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	12,000,000
12.	Conductors (4800 T @ 1600/T)	7,680,000
13.	Superstructure (Structural) 9500 T @ $4500	42,750,000
		$383,010,000

TABLE 6-18

CONCRETE GRAVITY STRUCTURE

NORTH ALEUTIAN SHELF - BASE CASE

(300' WATER DEPTH, 1,200,000 BARREL STORAGE)

FABRICATION COST ESTIMATE

1.	Graving Dock	$ 10,000,000
	Land Acquisition	
	Graving Dock Construction	
2.	Concrete (228,000 CY @ $960)	132,480,000
3.	Steel Skirts and Dowels	2,500,000
4.	Launching and Mooring	2,800,000
5.	Inserts and Fastenings	1,900,000
6.	Heavy Weight Ballast	1,600,000
7.	Support Facilities for Construction Afloat	4,700,000
8.	Mechanical Systems	42,000,000
9.	Electrical Systems	4,700,000
10.	Utility Shaft Equipment and Decks	27,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	11,000,000
12.	Conductors (4800 T @ 1600/T)	6,240,000
13.	Superstructure (Structural) 9500 T @ $4500	42,750,000
		$289,670,000

TABLE

ST. GEORGE OR NAVARIN BASIN - BASE CASE
TRANSPORTATION AND INSTALLATION COST ESTIMATE

ITEM		EQUIPMENT	DESCRIPTION	TIME	COST
1	Deliver modules to deck fabrication site and install on deck support structure	Local barge, heavy duty shear leg on crane barge	Modules fabricated in adjacent inland water in Puget Sound or Japan	Concurrent 120 days	$3,000,000
2	Install moorings at mating site	----	Anchors, chains, buoys	120 days	1,000,000
3	Transport deck to mating site	Large barges or tank mid-bodies	Tow in protected waters	20 days	3,500,000
4	Tow GBS concrete substructure to deck mating site and moor	Large domestic tugs	Tow in protected waters	10 days	500,000
5	Ballast down concrete substructure, mate deck, deballst, secure	----	----	15 days	1,500,000
6	Ocean tow to installation site	Requires 50000 to 80000 H.P. (5-6 tugs)	Tow in open sea Refueling required	40 days	5,000,000
7	Preset position markers, detailed bathymetry, clear site as necessary	Work and supply boats	----	30 days	2,500,000
8	Ballast structure down to sea floor, penetrate dowels and skirts	Tug boats	----	4 days	2,000,000
9	Underbase grouting	Barge or supply boats	Pumping sea water--cement grout under base 6000 m^3 to 8000 m^3	30 days	3,500,000
10	Removal of sand, ballast, drill out conductors plugs	Drill rig and air lifts		20 days	1,000,000
				TOTAL	23,500,000

TABLE 6-20

450' W.D. CONCRETE STRUCTURE

ST. GEORGE OR NAVARIN - BASE CASE

COST SUMMARY

(TO NEAREST MILLION)

Fabrication	383,000,000
Transportation and Installation	24,000,000
Home Office at 10%	41,000,000
Contingency at 30%	134,000,000
Total	$582,000,000

TABLE 6-21

300' W.D. CONCRETE STRUCTURE

NORTH ALEUTIAN SHELF - BASE CASE

COST SUMMARY

(TO NEAREST MILLION)

Fabrication	$290,000,000
Transportation and Installation	24,000,000
Home Office at 10%	31,000,000
Contingency at 30%	104,000,000
Total	$449,000,000

TABLE 6-22

BERING SEA CONCRETE STRUCTURE COST (1982 U.S. $\$x10^6$)

WATER DEPTH VARIATIONS - 300,000 BPD

	St. George Basin			Navarin Basin			N. Aleutian Shelf		
	None	1.2MM BBL	3MM BBL	None	1.2MM BBL	3MM BBL	None	1.2MM BBL	3MM BBL
Fabrication ..	313	383	474	313	383	474	231	290	353
Transp/Install.	24	24	24	24	24	24	24	24	24
Home Office @ 10%	34	41	50	34	41	50	26	31	38
Contingency @ 30%	111	134	164	111	134	164	84	104	125
TOTAL	482	582	712	482	582	712	365	449	540

TABLE 6-23

BERING SEA CONCRETE STRUCTURE COST (1982 U.S. $\$x10^6$)

STORAGE VARIATION - 300,000 BPD - 450' W.D.

	St. George Basin			Navarin Basin			N. Aleutian Shelf		
	None	1.2MM BBL	3MM BBL	None	1.2MM BBL	3MM BBL	None	1.2MM BBL	3MM BBL
Fabrication	313	383	474	313	383	474	231	290	353
Transp/Install.	24	24	24	24	24	24	24	24	24
Home Office @ 10%	34	41	50	34	41	50	26	31	38
Contingency @ 30%	111	134	164	111	134	164	84	104	125
TOTAL	482	582	712	482	582	712	365	449	540

TABLE 6-24

BERING SEA CONCRETE STRUCTURE COST (1982 U.S. $\$ \times 10^6$)

PRODUCTION VARIATION

450' W.D. - 1,200,000 BARREL STORAGE

	St. George Basin		Navarin Basin		N. Aleutian Shelf	
Production	100000	300000	100000	300000	100000	300000
Fabrication	357	383	357	383	265	290
Transp/Install.	24	24	24	24	24	24
Home Office @ 10%	38	41	38	41	29	31
Contingency @ 30%	126	134	126	134	95	104
TOTAL	545	582	545	582	413	449

TABLE 6-25

CONCRETE GRAVITY STRUCTURE

ST. GEORGE OR NAVARIN - 450' W.D.

WATER DEPTH VARIATION

FABRICATION COST ESTIMATE

		300'	600'
1.	Graving Dock	10,000,000	10,000,000
	Land Acquisition		
	Graving Dock Construction		
2.	Concrete (204,000 CY @ $960) (326,000 CY @ $960	195,840,000	312,960,000
3.	Steel Skirts and Dowels	4,000,000	4,000,000
4.	Launching and Mooring	2,800,000	2,800,000
5.	Inserts and Fastenings	1,900,000	1,900,000
6.	Heavy Weight Ballast	1,600,000	4,000,000
7.	Support Facilities for Construction Afloat	4,700,000	4,700,000
8.	Mechanical Systems	42,000,000	46,000,000
9.	Electrical Systems	4,700,000	4,700,000
10.	Utility Shaft Equipment and Decks	26,000,000	28,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	11,000,000	13,000,000
12.	Conductors (4800 T @ 1600/T)	7,680,000	7,680,000
13.	Superstructure (Structural) 9500 T @ $4500	42,750,000	42,750,000
		$345,970,000	$482,490,000

TABLE 6-26

CONCRETE GRAVITY STRUCTURE

ST. GEORGE OR NAVARIN - 450' W.D.

STORAGE VARIATION

FABRICATION COST ESTIMATE

		3 MILLION BARRELS	NO STORAGE
1.	Graving Dock	10,000,000	10,000,000
	Land Acquisition		
	Graving Dock Construction		
2.	Concrete (310,000 CY @ $960) (228,000 CY @ $960	297,600,000	196,080,000
3.	Steel Skirts and Dowels	4,000,000	4,000,000
4.	Launching and Mooring	2,800,000	2,800,000
5.	Inserts and Fastenings	1,900,000	1,900,000
6.	Heavy Weight Ballast	1,000,000	2,600,000
7.	Support Facilities for Construction Afloat	4,700,000	4,700,000
8.	Mechanical Systems	52,000,000	16,000,000
9.	Electrical Systems	4,700,000	2,000,000
10.	Utility Shaft Equipment and Decks	33,000,000	10,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	12,000,000	12,000,000
12.	Conductors (4800 T @ 1600/T)	7,680,000	7,680,000
13.	Superstructure (Structural) 9500 T @ $4500	42,750,000	42,750,000
		$474,130,000	$312,510,000

TABLE 6-27

CONCRETE GRAVITY STRUCTURE

NORTH ALEUTIAN SHELF

STORAGE VARIATION

FABRICATION COST ESTIMATE

		2 MILLION BARRELS	NO STORAGE
1.	Graving Dock	10,000,000	10,000,000
	Land Acquisition		
	Graving Dock Construction		
2.	Concrete (190,000 CY @ $960) (138,000 CY @ $960	182,400,000	118,680,000
3.	Steel Skirts and Dowels	2,500,000	2,500,000
4.	Launching and Mooring	2,800,000	2,800,000
5.	Inserts and Fastenings	1,900,000	1,900,000
6.	Heavy Weight Ballast	1,000,000	1,600,000
7.	Support Facilities for Construction Afloat	4,700,000	4,700,000
8.	Mechanical Systems	50,000,000	14,000,000
9.	Electrical Systems	4,700,000	2,000,000
10.	Utility Shaft Equipment and Decks	33,000,000	10,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	11,000,000	11,000,000
12.	Conductors (3900 T @ 1600/T)	6,240,000	6,240,000
13.	Superstructure (Structural) 9500 T @ $4500	42,750,000	42,750,000
		$352,990,000	$230,870,000

TABLE 6-28

CONCRETE GRAVITY STRUCTURE

ST. GEORGE OR NAVARIN - 450' W.D.

PRODUCTION VARIATION (100,000 BPD)

FABRICATION COST ESTIMATE

Item	Cost
1. Graving Dock	$ 10,000,000
Land Acquisition	
Graving Dock Construction	
2. Concrete (228,000 CY @ $960)	218,880,000
3. Steel Skirts and Dowels	4,000,000
4. Launching and Mooring	2,800,000
5. Inserts and Fastenings	1,900,000
6. Heavy Weight Ballast	1,600,000
7. Support Facilities for Construction Afloat	4,700,000
8. Mechanical Systems	42,000,000
9. Electrical Systems	4,700,000
10. Utility Shaft Equipment and Decks	27,000,000
11. Riser and Drilling Shaft Equipment and Conductor Guides	8,000,000
12. Conductors (3000 T @ 1600/T)	4,800,000
13. Superstructure (Structural) 9500 T @ $4500	27,000,000
	$357,380,000

TABLE 6-29

CONCRETE GRAVITY STRUCTURE

NORTH ALEUTIAN SHELF

PRODUCTION VARIATION (100,000 BPD)

FABRICATION COST ESTIMATE

	Item	Cost
1.	Graving Dock	$ 10,000,000
	Land Acquisition	
	Graving Dock Construction	
2.	Concrete (228,000 CY @ $960)	132,480,000
3.	Steel Skirts and Dowels	2,500,000
4.	Launching and Mooring	2,800,000
5.	Inserts and Fastenings	1,900,000
6.	Heavy Weight Ballast	1,000,000
7.	Support Facilities for Construction Afloat	4,700,000
8.	Mechanical Systems	40,000,000
9.	Electrical Systems	4,700,000
10.	Utility Shaft Equipment and Decks	27,000,000
11.	Riser and Drilling Shaft Equipment and Conductor Guides	7,000,000
12.	Conductors (2400 T @ 1600/T)	3,840,000
13.	Superstructure (Structural) 9500 T @ $4500	27,000,000
		$264,920,000

PROPRIETARY INFORMATION UNTILL MAY 1, 1987

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

FINAL REPORT

TABLE OF CONTENTS

Detailed indices are included in each Section.

LIST OF FIGURES

SECTION 7.0

7.0 PIPELINE

7.1 Scope of Work

A submarine pipeline from the production platform to a shore terminal is an alternative to the use of tankers transporting crude oil from an offshore loading terminal. The scope of work for this conceptual design of the submarine pipelines is to:

- Size diameters and wall thicknesses of the pipelines for transportation of 100 MBPD and 300 MBPD of crude oil from the North Aleutian, St. George and Navarin Basins to a shore terminal.
- Select a suitable grade of pipe steel for the submarine pipelines.
- Identify pipeline components, construction procedures and construction equipment appropriate to Bering Sea environmental conditions.
- Identify engineering problems or constraints to development of a submarine pipeline.
- Develop order of magnitude costs for representative pipelines for purposes of pre-lease planning.

7.2 Design Criteria and Assumptions

- The pipelines will be conceptually designed in compliance with ANSI B31.4-1974, "Liquid Petroleum Transportation Piping Systems," and MMS-OCS Order No. 9 for the Bering Sea.
- Pipeline design and construction practices used in the North Sea will be applied to the Bering Sea.
- For construction planning, the weather window and climactic severity during the working season are assumed to be the same in the Bering Sea as in the North Sea.
- The crude is assumed sweet with an API gravity between 25 and 35.

- The pour point temperature of the crude-oil/gas-liquids mixture transported is less than 35°F, and the mixture will flow at a temperature of 35°F over the whole length of pipeline. That is, hot oil cool-down will not be taken into account for the sake of conservatism.
- The production will be dry (water removed) at the production platform and will flow as a single liquid phase.
- The pipeline diameters and wall thicknesses will be selected on the assumption that the pipelines will conservatively transport the nominal 100 and 300 MBPD base-case production rates and throughput can be increased in the future by increasing the pumps discharge pressure.
- The maximum water depth is 600 feet.
- The length of the submarine pipeline from each of the three offshore lease basins is estimated by assuming the production platforms to be at about the center of each basin and that all pipelines terminate at Lefthand Bay (Figure 7-1).

7.3 Pipeline Design

7.3.1 Selection of Pipe Steel and Pipe Dimensions

With no present knowledge of the properties of the oil in the Bering Sea lease areas, especially its viscosity at the flow temperature, the pipeline diameters and wall thicknesses selected here are liable to substantial revision. However, guided by these results, which are thought to be conservative, construction procedures and cost estimates can be adjusted by later design revisions when the properties of the crude become known.

Current practice in laying deep water pipelines favors the use of high strength steel. High stresses sustained in bending and external hydrostatic pressure would otherwise require the pipe to have a thick wall with a disproportionate increase in the cost of the steel. API-5LX-60 steel has found frequent application in North Sea pipelines.

The pipe diameter to thickness ratio, D/t, customarily varies in the range of 40 to 43 for water depths of 300 to 500 feet typical of the platform locations in the North Sea. The D/t ratio is not necessarily determined by the collapse pressure of the pipe when subjected to combined bending and external hydrostatic pressure. For the long, North Sea pipelines laid over distances of 100 nautical miles and more, the controlling criterion is often the allowable internal pressure. The capability of increasing the discharge pressure of the mainline pumps may eliminate the need for an intermediate booster station or may permit increased throughput as platform production rates expand. This philosophy will be used in the conceptual design of pipelines for the Bering Sea. Later, specific design may revise the wall thickness downwards with a corresponding decrease in material cost, but in a conceptual study such as this, the conservative position regarding uncertainties is recommended.

The steel grade will be taken to be API-5LX-65. This grade is equivalent in weldability to API-5LX-60, and the cost differential is small compared to the 5000 psi gain in minimum yield strength.

The selected pipe dimensions for the two cases of throughput are summarized below. These pipe dimensions will apply for all three lease basins to allow comparison among the six combinations of throughput and distance in terms of the construction equipment required.

Throughput	Outside Diameter (inches)	Wall Thickness (inches)	D/T	Allowable Working Pressure (Approximate) (psi)	Mean Flow Speed (ft/sec.)
100 MBPD	24	0.625	38.4	2438	2.3
300M BPD	36	0.875	41.1	2275	3.0

As described above, the pipewall thicknesses chosen are somewhat greater than needed to withstand combined stresses in the sagbend during lay. With a D/t of 40, API-5LX-65 pipe of 1% ovality could be laid in water depths of about 900 feet. The extra wall thickness, however, produces an allowable working pressure, based upon 72% of minimum yield strength, of over 2000 psi.

The pipe diameters are selected to produce low values of mean flow velocity in both cases of throughput. This is done in anticipation of a possibly high oil viscosity which produces a high wall shear stress and a steep hydraulic gradient. Should it be found that the oil viscosity is very low, then even the high throughput of 300 MBPD could be put through the 24" O.D. pipe at a speed of 7 ft/sec - well within the accepted speed range.

The range of oil viscosities can be estimated from the assumed API gravity range of 25°-35°. By use of Beal's correlation of viscosity among 655 gas-free, crude oil samples at 100°F and at atmospheric pressure* together with an exponential function for viscosity variation with temperature, the following table was constructed.

* Petroleum Production Handbook, Vol. II; Millet the Printer, Inc., 2nd Ed., 1962.

7.3.2 Pipe Coatings and Anodes

The pipeline will not be coated internally but will be externally coated to protect it from corrosion. This coating could be a mastic, coal tar material with an external wrap or one of the many newer coatings that have become commercially available. A fusion-bonded epoxy coating has become popular for submarine pipeline applications, especially when no concrete weight coating is to be used.

The pipe joints will be given a final coating of concrete reinforced by steel mesh to provide enough negative buoyancy to stabilize the pipe on the seabed against the hydrodynamic forces of waves and currents. Concrete with a weight density of 140 lbs/ft^3 is commonly used for this purpose. The weight coating also protects the pipe and its corrosion coating against damage from the jet trenchers, if the pipe is to be trenched, and from trawl boards where the pipe is left exposed on the seabed.

For a concrete weight density of 140 lbs/ft^3, the 24" O.D. pipe will require a concrete thickness of about 1-1/2 inches and the 36" O.D. pipe, a thickness of about 2-1/4 inches to provide stability against drag and lift forces.

The concrete coating can be applied by spinning it on the pipe or by casting it. The latter method gives much closer control of weight and dimension tolerances as well as greater concrete strength. The cast method was used in the cost estimate.

It is expected that the pipe joints will be shipped to the Bering Sea area fully wrapped and coated with only a short length of pipe at each end exposed to permit making the field-joint weld on the laybarge. Once the field joint has been made and and has passed x-ray inspection, the coating gap is filled with mastic at the barge's doping station.

The pipe coating yard would be set up somewhere near a port where the pipe can be loaded for shipment to Alaskan waters. No further onshore work would be required on the pipe at the offshore construction support base. The pipe joints would just be delivered there for loading on pipehaulers which supply the laybarges. An exception may occur in pipe deliveries to 3rd generation barges. For some of these, the preference is to handle double joints of pipe. It may be desirable to establish a double-jointing yard at the supply base, and space is available for such work as can be seen in the layout of the resupply base (Figure 7-6).

To protect the pipeline against electrolytic corrosion, sacrificial anodes of zinc or aluminum will be attached to the pipe at regularly spaced intervals . These should be flush with the outside of the concrete coating to avoid being damaged while passing over the laybarge's stinger, and they must be in good electrical contact with the pipe steel.

7.3.3 Buckle Arrestors

Buckles occasionally occur during or even after construction of a submarine pipeline. In shallow water, the buckle remains localized at its point of development, but at some depth determined by the pipe steel strength and wall thickness, the buckle may propagate along the pipe. Propagation can continue until the external hydrostatic pressure drops to a magnitude which cannot create the typical buckle geometry of a "dog-bone" pipe cross-section. This propagation distance could be many miles over the already-laid pipe depending on how much pipe had been laid in water depths exceeding the critical depth for the inception of buckle propagation.

LIST OF FIGURES

SECTION 7.0

TABLE OF CONTENTS

SECTION 7.0

have a thick wall with a disproportionate increase in the cost of the steel. API-5LX-60 steel has found frequent application in North Sea pipelines.

The pipe diameter to thickness ratio, D/t, customarily varies in the range of 40 to 43 for water depths of 300 to 500 feet typical of the platform locations in the North Sea. The D/t ratio is not necessarily determined by the collapse pressure of the pipe when subjected to combined bending and external hydrostatic pressure. For the long, North Sea pipelines laid over distances of 100 nautical miles and more, the controlling criterion is often the allowable internal pressure. The capability of increasing the discharge pressure of the mainline pumps may eliminate the need for an intermediate booster station or may permit increased throughput as platform production rates expand. This philosophy will be used in the conceptual design of pipelines for the Bering Sea. Later, specific design may revise the wall thickness downwards with a corresponding decrease in material cost, but in a conceptual study such as this, the conservative position regarding uncertainties is recommended.

The steel grade will be taken to be API-5LX-65. This grade is equivalent in weldability to API-5LX-60, and the cost differential is small compared to the 5000 psi gain in minimum yield strength.

The selected pipe dimensions for the two cases of throughput are summarized below. These pipe dimensions will apply for all three lease basins to allow comparison among the six combinations of throughput and distance in terms of the construction equipment required.

Throughput	Outside Diameter (inches)	Wall Thickness (inches)	D/T	Allowable Working Pressure (Approximate) (psi)	Mean Flow Speed (ft/sec.)
100 MBPD	24	0.625	38.4	2438	2.3
300M BPD	36	0.875	41.1	2275	3.0

As described above, the pipewall thicknesses chosen are somewhat greater than needed to withstand combined stresses in the sagbend during lay. With a D/t of 40, API-5LX-65 pipe of 1% ovality could be laid in water depths of about 900 feet. The extra wall thickness, however, produces an allowable working pressure, based upon 72% of minimum yield strength, of over 2000 psi.

The pipe diameters are selected to produce low values of mean flow velocity in both cases of throughput. This is done in anticipation of a possibly high oil viscosity which produces a high wall shear stress and a steep hydraulic gradient. Should it be found that the oil viscosity is very low, then even the high throughput of 300 MBPD could be put through the 24" O.D. pipe at a speed of 7 ft/sec - well within the accepted speed range.

The range of oil viscosities can be estimated from the assumed API gravity range of 25°-35°. By use of Beal's correlation of viscosity among 655 gas-free, crude oil samples at 100°F and at atmospheric pressure* together with an exponential function for viscosity variation with temperature, the following table was constructed.

* Petroleum Production Handbook, Vol. II; Millet the Printer, Inc., 2nd Ed., 1962.

Crude Oil Gravity (°API)	Kinematic Viscosity (Centistokes) 100°F	60°F	35°F
20	128	640	1800
25	41	190	450
30	15	55	110
35	7	18	30
40	4	8	11

At the ambient seawater temperature, assumed to be 35°F, the highest kinematic viscosity expected is 450 centistokes and the lowest, 30 centistokes. Both estimates will be lowered with the inclusion of natural gas liquids in the stream, but not appreciably.

The estimated pipeline lengths from each of the three assumed platform locations are given in the table below.

PIPELINE DISTANCES
(Nautical Miles Unless Otherwise Specified)

		N. Aleutian	St. George	Navarin
Lefthand Bay Terminal to Herendeen Bay	16	-	-	-
Herendeen Bay to the Cape Rozhnof area	23	-	-	-
Cape Rozhnof area to the Offshore Platform	-	26	203	572
Total Distance (Naut. mi.)		65	242	611
Total Distance (Stat. mi.)		75	276	700

The ground profile and hydraulic gradients from the postulated platform in the St. George Basin to Lefthand Bay are shown in Figures 7-2 and 7-3 . In the first figure, the case of 100 MBPD throughput for a 24 inch O.D. pipeline is shown. The kinematic viscosity is assumed to be 110 centistokes from the center of the API gravity range in the preceding table. The pump discharge head at the platform is seen to be just about half the allowable giving a significant margin for increased throughput or higher viscosity.

In Figure 7-3, the 300 MBPD case of throughput in a 36-inch O.D. pipe is illustrated for the same platform location but with the highest expected crude viscosity. Still, the discharge head is about half the allowable.

It can be judged from these figures that the same pipeline sizes from the North Aleutian platform to Lefthand Bay, a distance of only 75 statute miles, are much too large for the intended throughputs. This is simply a function of site location; for if the terminal were situated at Kupreanof harbor, say, the crossing of high ridges well downstream of the pipeline route would cause a rise in the required mainline-pump discharge head while the length of the sea line would remain at 26 nautical miles. Therefore, the same pipeline diameters are used for the North Aleutian as for the more distant basins for the two production cases.

The ground profile and hydraulic gradients from the most distant platform, the one in the Navarin Basin, are shown in Figures 7-4 and 7-5 - one for the 24-inch O.D. line at 100 MBPD and one for the 36-inch O.D. line at 300 MBPD. A mean kinematic viscosity of 110 centistokes is used in both cases. For oil of this viscosity, the allowable internal pressure for both pipeline sizes is less than the pressure required to obtain the nominal throughputs. For this reason, an intermediate booster station is required as described in Section 7.4.

7.3.2 Pipe Coatings and Anodes

The pipeline will not be coated internally but will be externally coated to protect it from corrosion. This coating could be a mastic, coal tar material with an external wrap or one of the many newer coatings that have become commercially available. A fusion-bonded epoxy coating has become popular for submarine pipeline applications, especially when no concrete weight coating is to be used.

The pipe joints will be given a final coating of concrete reinforced by steel mesh to provide enough negative buoyancy to stabilize the pipe on the seabed against the hydrodynamic forces of waves and currents. Concrete with a weight density of 140 lbs/ft^3 is commonly used for this purpose. The weight coating also protects the pipe and its corrosion coating against damage from the jet trenchers, if the pipe is to be trenched, and from trawl boards where the pipe is left exposed on the seabed.

For a concrete weight density of 140 lbs/ft^3, the 24" O.D. pipe will require a concrete thickness of about 1-1/2 inches and the 36" O.D. pipe, a thickness of about 2-1/4 inches to provide stability against drag and lift forces.

The concrete coating can be applied by spinning it on the pipe or by casting it. The latter method gives much closer control of weight and dimension tolerances as well as greater concrete strength. The cast method was used in the cost estimate.

It is expected that the pipe joints will be shipped to the Bering Sea area fully wrapped and coated with only a short length of pipe at each end exposed to permit making the field-joint weld on the laybarge. Once the field joint has been made and and has passed x-ray inspection, the coating gap is filled with mastic at the barge's doping station.

The pipe coating yard would be set up somewhere near a port where the pipe can be loaded for shipment to Alaskan waters. No further onshore work would be required on the pipe at the offshore construction support base. The pipe joints would just be delivered there for loading on pipehaulers which supply the laybarges. An exception may occur in pipe deliveries to 3rd generation barges. For some of these, the preference is to handle double joints of pipe. It may be desirable to establish a double-jointing yard at the supply base, and space is available for such work as can be seen in the layout of the resupply base (Figure 7-6).

To protect the pipeline against electrolytic corrosion, sacrificial anodes of zinc or aluminum will be attached to the pipe at regularly spaced intervals . These should be flush with the outside of the concrete coating to avoid being damaged while passing over the laybarge's stinger, and they must be in good electrical contact with the pipe steel.

7.3.3 Buckle Arrestors

Buckles occasionally occur during or even after construction of a submarine pipeline. In shallow water, the buckle remains localized at its point of development, but at some depth determined by the pipe steel strength and wall thickness, the buckle may propagate along the pipe. Propagation can continue until the external hydrostatic pressure drops to a magnitude which cannot create the typical buckle geometry of a "dog-bone" pipe cross-section. This propagation distance could be many miles over the already-laid pipe depending on how much pipe had been laid in water depths exceeding the critical depth for the inception of buckle propagation.

Rather than increase the pipe wall thickness to prevent buckle propagation, which could lead to considerable expense for material, the practice is to place thick, steel bracelets at intervals along the pipe. These bracelets or buckle arrestors can be welded to the pipe or simply slip-fitted in place. Their function is to prevent the pipe from developing a "dog-bone" cross-section and thus to halt the progress of the buckle.

The buckle arrestors are placed on every other or even every third joint; so that at most, only three joints of pipe will be lost in the event a buckle propagates. Buckle arrestors, like the sacrificial anodes, are flush with the concrete coating.

As a rule-of-thumb in the North Sea, buckle arrestors are used on pipe in water depths greater than about 350 feet.

7.4 Booster Station

In the particular case of a submarine pipeline from the Navarin basin to the Alaska Peninsula, the line length is so great than an intermediate booster station is recommended. This is situated at about the halfway point of the pipeline in order to balance the overall horsepower requirements between the production platform pumps and those at the booster station.

7.4.1 Topsides

The flow diagram for the booster station is shown in Figure 7-7 .

The station is normally not manned. Facilities are available to accommodate several persons during periods of maintenance and repair.

Fuel gas for the pumps is extracted from a side stream of the mainline flow.

No surge tankage is required at the booster station. Should the pumps shut down for any reason, the fluid pressure will drop on the discharge side of the pumps allowing a check valve to open on the bypass line. This function will take place automatically. The pumps will be equipped with variable speed drives so that during startup, the discharge pressure will be gradually built up until the check valve on the bypass closes and all the mainline flow is once again directed through the pumps.

Under booster-station bypass conditions and with the production platform pump-discharge head held constant, the pipeline throughput would drop about 25% for either the 24-inch O.D. or 36-inch O.D. pipeline systems.

7.4.2 Structure

The booster station would be located to the northeast of the Pribilof islands in about 200-250 feet of water. The structure was not detailed during this study, but the following items were assumed for purposes of cost estimating.

- The structure could be either a steel pile supported or concrete gravity platform. A steel structure is assumed for cost estimating.

- The structure would be significantly simpler and lighter than the drilling and production platforms because of the assumed lower payload and no requirements for housing wells within the structure.

- The following allowances are made for cost estimating:

Jacket Weight - 3000 T
Piling Weight - 2500 T
Installation Time - 30 days*

* Assuming the work is done by either one of the pipelay barges if it is a combination vessel or by mobilizing a 800-1000 T derrick barge from the Pacific Basin.

7.5 Onshore Pipeline

The pipeline section from the south end of Herendeen Bay to the vicinity of the shoreline on the Bering Sea, a distance of 23 nautical miles, is treated as a submarine pipeline. Its construction is described in Section 7.7.6.

The actual onshore pipeline construction takes place over 18.5 statute miles of right-of-way from Herendeen Bay to Lefthand Bay. The pipe is assumed to be of the same steel and wall thickness as the submarine portion for construction cost estimation. In the final design, the wall thickness and steel grade of the land line may be reduced for the sake of economy.

The land line will have a corrosion coating but no concrete coating.

It may be preferred to use an induced current method of cathodic protection for this section of line, and it will be electrically insulated from the submarine section.

The land line would be trenched and buried over its whole length. An allowance is made in the cost estimate for drilling and shooting part of the route.

7.6 Pipeline Communications

A microwave communication system would be most suitable for the North Aleutian Basin production platform, since it is postulated to be within 26 miles of a shore-based repeater station. However, both the St. George and Navarin Basin platforms are too far distant from land to use a microwave transmission system.

A hardwire system is assumed for cost estimating purposes for all three platform locations, but consideration should be given to substituting a microwave system for the North Aleutian platform and the ALASCOM satellite system for the more distant platforms.

7.7 Construction Planning

7.7.1 Pipeline Schedule Constraints

Pipeline construction is not anticipated to be on the critical path to completion of the entire production/transportation system. Even the longest line, the one from the Navarin Basin, can be completed in two working seasons, the 3rd and 4th summer season from the start of the development (see Schedule, Section 10). However, the platform structures are not scheduled for installation offshore until the end of the 4th year. This means that tie-in of the pipeline to the platform riser will be delayed until the working season of the 5th year. A laybarge is the preferred vessel for the platform tie-in operation, but maintaining a 3rd generation laybarge for three consecutive years is costly. It is probably less expensive to demobilize the barge at the end of pipelay operations in the second season of lay and to combine into one workbarge the final construction activities of:

- pipeline tie-in to the production platform
- final hydro-test of the completed line

Moving the pipeline construction schedule back to the 4th and 5th working seasons is not thought to be an acceptable alternative, for the pipeline then becomes the critical path. If, for any reason, such as severe weather or barge breakdown, the line is not laid in the last season, the work is delayed a full year into the summer of the 6th year.

7.7.2 Lay Barges

Lay barges that have operated in the North Sea are usually catalogued into two classes, 2nd generation and 3rd generation. Small barges which would belong to the early or 1st generation class have not been used in the severe climate of the North Sea.

By the time the first major pipeline was laid in the North Sea, barge development had reached the 2nd generation stage. This first line was a 16" O.D. gas pipeline laid by the Brown and Root barge, Hugh W. Gordon. The year was 1966, and the pipeline connected British Petroleum's West Sole Field to a plant located at Easington, Yorkshire.

In the late 1960's and early 1970's, 2nd generation lay barge work proliferated in the North Sea; and new barges were built, not only to serve that market but also to lay pipe for developments in east and southeast Asia.

For the most part, 2nd generation lay barges are flat-bottomed, square-ended vessels without self-propulsion and with overall lengths between 400 and 460 feet. The barges hold position on multiple anchor lines and advance pipe-joint by pipe-joint by winching ahead on the forward anchor lines.

Single concrete-coated pipe joints are delivered to the barge by pipe-haul, supply boats. The pipe is stalked on the pipeway at the head of the barge; and, as the barge advances, every joint of pipe passes through each of several welding stations, an X-ray inspection station and a field-joint coating station.

Welding may be done manually by welders or automatically by using one of the several types of automatic welding machines now on the market. By the time a pipe joint reaches the stern of the barge, all work on the field joint is complete, and the joint passes over the stinger overbend control enroute to the seabed.

2nd generation barges suited to North Sea or Bering Sea conditions are listed in the following table:

2ND GENERATION LAY BARGES
FOR THE BERING SEA

BROWN AND ROOT	McDERMOTT
BAR 323	LB - 23
BAR 324	LB - 27
L. B. MEADERS	LB - 28
HUGH W. GORDON	LB - 29

SAIPEM	OTHER
CASTORO II	ETPM 701
CASTORO V	CHOCTAW II
	KUROSHIO II
	KOKAN PIONEER I
	MV ORCA

Most of these barges have seen North Sea service, but some, like the Kokan Pioneer I and Kuroshio II, have been operating in Asian waters only.

All units listed are flat-bottomed, blunt-ended barges except the ETPM 701 and MV ORCA. These are self-propelled, pipe-laying and heavy lift-crane ships. The Choctaw II is a twin-hulled, column stabilized vessel (semi-submersible).

The list of 2nd generation barges is not complete. There are many others - most of which are too small or too lightly structured for North sea work. Some others may be functional in severe climates, but confirmatory data are not at hand for example, the BAR 332.

Finally,there is a group of derrick vessels which could be modified for pipelaying. These include:

- The Netherlands Offshore Company's Blue Whale and Sea Lion I.
  These, like the MV ORCA, are cargo vessels modified to lay pipe and perform heavy lifts at sea.
- The Ocean Builder I, a ship-shaped, derrick vessel.
- McDermott's LB Nos. 14,15, 17,20,21,25 and 26.

The primary limitation recognized in 2nd generation barges is susceptability to rough seas. Typically, pipe lay stops in significant waveheights of 6 feet on the beam and 8 feet on the bow. Worsening weather requires dropping the pipe and stinger,and storm conditions often require that the barge be towed to shelter.

The variability of North Sea weather from year to year can have a major impact on the expected performance of 2nd generation barges. 1976 was a poor year in the North Sea. It was reported that a persistent swell occurred throughout the summer, and 2nd generation laybarge production declined accordingly.

Limitations in the 2nd generation barges inspired the development of a 3rd generation which came on the market in the middle of the 1970's. There are only five of these operating and they have one quality in common - great size. The five are listed below:

3RD GENERATION LAY BARGES

SEMAC I
LB 200 (formerly Viking Piper)
CASTORO VI
BAR 347
ETPM 1601

Three of the 3rd generation lay barges are semi-submersibles, the Semac I, LB 200 and Castoro VI. This hull configuration provides much greater seaworthiness, and it is claimed that these vessels can continue pipe lay in significant waves of 15 to 17 feet. In the North Sea, this kind of performance triples the average rate of seasonal production delivered by 2nd generation barges, and all but eliminates the effect on production of summer weather variability. However, the barge's ability to continue pipelay in high seas creates a different constraint - the inability to resupply pipe to the barge in heavy weather. If bad weather persists, the barge uses up the deck supply of pipe and must wait for seas to subside enough to bring more pipe aboard.

By their nature, semi-submersible hulls lack the capability to carry heavy deck loads. Pipe-load capacity for these three barges range from 6000 tons for the Castoro VI to 8000 tons for the LB 200.

The BAR 347 is a flat bottomed barge 650 feet in length. This hull configuration allows it to have a deck capacity of 20,000 tons of pipe, but the same hull shape restricts its working tolerance to waves well below that of the semi-submersibles.

The great length of the BAR 347 allows welding double-jointed pipe at 6 weld stations. This is the same number of weld stations typical of 2nd generation barges handling single-jointed pipe. Even if weather sensitivity were the same for the BAR 347 as its smaller, 2nd generation counterparts, its production rate would double theirs.

The ETPM 1601 has a ship-shape hull 628 feet long. It, too, double joints but at 5 welding stations. This vessel ceases work in significant wave heights over 10 feet. These features give it a mean production rate somewhere intermediate between the 3rd generation semi-submersibles and the 2nd generation barges.

7.7.3 Lay Barge Requirement for the Bering Sea

Data on wave persistence, wave height recurrence and variability from year to year are not available for the area of interest in the Southern Bering Sea. It is assumed, for planning purposes, that these quantities are the same as for the North Sea. Moreover, the weather window for pipe lay operations in the Bering Sea is taken to be the same as for the North Sea: the six months from the beginning of April through September. In some years, ice cover might delay the start of the season.

For several reasons, emphasis is placed on the use of 2nd generation barges in preference to 3rd generation barges in construction planning of the several combinations of pipeline routes and pipe sizes. These reasons are:

- More 2nd generation barges are available.
- It is easier to mobilize 2nd generation barges to the Bering Sea.
- Two of the three pipeline distances are short enough to warrant the use of 2nd generation barges.

The great distance of the Navarin Basin production site from shore requires that only 3rd generation, semi-submersible barges be considered for this pipelay, because six or more 2nd generation barges would be needed to construct the pipeline in two seasons. Two such barges are presently in the North Sea area and one is in the Mediterranean. Two of the three would be needed for two consecutive seasons to construct either the 24" O.D. or 36" O.D. pipelines from the Navarin Basin to Cape Rozhnof on the north coast of the Alaska Peninsula.

These barges are too large to pass the Panama Canal and would have to be towed around Cape Horn. The one way distance from the North Sea to Dutch Harbor is estimated to be over 16,000 nautical miles.

Not only is it questionable whether two of the three semi-submersibles would be available for work in the Bering Sea, but the cost of mobilizing, demobilizing and retaining these vessels between seasons would be considerable. The alternative is to construct two new semi-submersible lay barges somewhere on the Pacific Rim. This option is actually the one cost estimated in Section 10.2. The total capital expenditure is about the same in either option, but the cash flow diagrams are different.

A pipeline from the St. George Basin to Cape Rozhnof can be laid using not more than three 2nd generation barges. Enough of these barges are accessible to the Pacific Ocean that there is a good expectation of obtaining them in time to meet the construction schedule. The Kuroshio II and Kokan Pioneer I are in Asiatic waters, and the BÀR 347 is in Southeast Asia. Use of this last barge could reduce the total number required for the St. George Basin pipeline construction from three to two; however, three were used for cost estimating purposes.

The Panama Canal permits a maximum vessel width of 106 ft. All the Brown and Root 2nd generation barges listed in the earlier table have a width of 100 feet which allows them to pass the canal. Any of these barges, if available in the Gulf of Mexico, could be mobilized.

The Choctaw II has a width of 106 feet, it, too, could be mobilized from the gulf through the canal, since it has made that passage before.

All of the McDermott barges measure 128 feet in width and would have to be mobilized around Cape Horn.

7.7.4 Lay Barge Production Rates

Under the assumptions stated previously that the weather window and the mean sea conditions are the same in the Bering Sea as in the North Sea, a mean total pipelay per working season can be assigned to the barge types, and these assignments are given in the table below:

Total Average Pipelay in One
6-Month Working Season (Nautical Miles)

	24" O.D. Pipe	36" O.D. Pipe
3rd generation, semi-submersible lay barge	216	143
2nd generation, flat-bottomed lay barge	72	48

The ETPM 1601 and BAR 347 are excluded from the table. If either were to be obtained, it would be for use on the St. George or North Aleutian pipelines, and the estimate for total average lay in one season would be increased accordingly.

From these average lay rates and the pipeline lengths assumed for each basin, it is a simple matter to determine the number of barges needed for each location and each pipe diameter. A substantial excess lay time allowance must be made for deviations from the mean production rate and for the barges to have time during the working season to participate in the pipeline shore connections and offshore pipeline tie-ins.

A laybarge must be used for the pipeline shore landing. Separate work barges are provided in the cost estimate to perform pipeline tie-ins offshore, but lay barges are usually preferred for this work. Laybarges are equipped with the necessary tools: anchor winches, davits, cranes and a diving spread. If the laybarges complete the scheduled pipe lay early, they could be diverted to assist in pipeline tie-ins, buckle repairs and hydrostatic testing.

Laybarge fleet requirements are given in the following table:

Pipeline	24" O.D. Pipe				36" O.D. Pipe			
	1st Season		2nd Season		1st Season		2nd Season	
	No.	Barge Type	No.	Barge Type	No.	Barge Type	No.	Barge Type
Navarin Basin to Shore	2	3rd Gen.	2	3rd Gen.	2	3rd Gen.	2	3rd Gen.
					1	2nd Gen.		
St. George Basin to Shore	2	2nd Gen.	2	2nd Gen.	3	2nd Gen.	2	2nd Gen.
North Aleutian Basin to Shore	1	2nd Gen.		—	1	2nd Gen.		—

The lay barge fleet selections tabulated above are summarized in the following subsections.

7.7.4.1 24" O.D. Submarine Pipeline Construction

Navarin Basin - three 3rd generation barge work seasons allow the laying of 648 nautical miles of pipe. This gives an excess lay capacity of 76 nautical miles or 13% over the 572 nautical miles required. This is not only a marginal difference to provide for variations in average lay rates, but it allows no time for pipeline shore connections. The retention of both 3rd generation barges for the second season is assumed to be a necessity.

St. George Basin - two second generation barges for two consecutive working seasons are expected to have a lay capacity of 288 nautical miles. This is an excess capacity of 85 nautical miles or 42% over the 203 nautical miles required. For planning purposes, this should be sufficient to accommodate other barge work like the shore connection and allow for pipelay production variability.

North Aleutian Basin - the 26 nautical miles of pipeline can be laid by one 2nd generation barge in one season, and this barge can complete shore landings and tie-ins in the same season.

Working conditions on this route are favored by the proximity of the Alaska Peninsula which gives shelter from the southerly winds prevailing in the working season.

7.7.4.2 36" O.D. Submarine Pipeline Construction

Navarin Basin - two 3rd generation barges working in two consecutive seasons (4 lay barge seasons), each at the average rate of 143 nm/season, lay exactly the total distance of 572 nautical miles. A third barge will be needed. This is designated to be a 2nd generation barge which would be deployed to construct the shore landing at the beginning of the first season of pipelay. It would then continue to lay pipe for as long as weather permits.

If the pipelay schedule is well advanced by the end of the first season, the 2nd generation barge can be demobilized. If not, this barge must be retained through a second season.

Every effort should be made to advance the work in the first season of lay. Carry over of the third barge into a second season not only creates more expense in barge operations but also in support operations, in additional subsea connections and in hydrostatic testing. Schedule advancement will be discussed further in the section on the pipeline shore landing, but, as suggested by the table above, cost estimation is based on demobilizing the 2nd generation barge at the end of the first season of lay.

St. George Basin - two 2nd generation barges working for two seasons together with a third 2nd generation barge in the first season have a marginal excess capacity of 37 nautical miles or 18%. As in the lay of a 36" O.D. pipeline to the Navarin Basin discussed above, effort should be made to increase the production in the first season, or the third barge must be retained for a second season.

North Aleutian Basin - as with the 24" O.D. line, one 2nd generation barge can lay the 36" O.D. line over 26 nm in one season and complete the shore connection as well.

7.7.5 Laybarge Support Vessels

Every laybarge is accompanied by two anchor-handling tugs and a survey vessel. For Bering Sea operations, it would be advisable to have the survey vessel also equipped for anchor handling. Additional tugs are maintained for station relief and crew changes.

Pipehaulers for the St. George or Navarin construction should be of the large size, with a deck capacity of 1000 long tons or more, as are currently in use or being constructed to supply the large barges. Their numbers depend on the distance of travel required for a round trip from the supply base and on the rate of pipe consumption by the barge.

Pipe inspection would be carried out by submarines supported by a tender. When two or more barges are working within reasonable distances of each other, one tender and its submarines may be shared between two barges. For example, when 3 barges are laying pipe on the route to the St. George Basin, two submarine tenders can support them.

7.7.6 Pipeline Shore Connections

The shore is low-lying and composed of loose materials along the entire north side of the Alaska Peninsula. The same procedure for bringing the pipeline ashore should apply to whichever onshore facility site is selected for development.

Generally, a lay barge can work with shore-based equipment to pull pipe through the surf zone in one of three ways:

- Pipe is joined on the lay barge in the usual way and pulled to shore using a shore-based winch.
- Pipe can be made up in long sections on shore ready for transfer to a pipeway. The Abandonment and Recovery (A & R) winch on the laybarge pulls each length into the sea. Then another section of pipe is placed on the pipeway and welded into the line.
- A large diameter sheave can be anchored on shore. A cable is passed from the lay barge's A & R winch through the sheave and back to a pulling head on the first joint of pipe at the laybarge. Using its A & R winch, the laybarge can make its own pipe pull to shore.

Along much of the Alaska Peninsula, the lack of roads and infrastructure would make it difficult to land pipe for assembly onshore. The first pipe pull method in the list above is probably favored for most locations.

Among the alternative ways of arranging for pipe burial through the surf zone, the method deemed appropriate for the north shore of the Alaska Peninsula is sheet piling and trestle construction. An open trestle is constructed from shore, and sheet piling is driven and braced on either side of the trestle as the work extends seaward. The sheet piling protects the trestle from storm breakers and keeps the pipeline trench, which is excavated from the trestle, from backfilling with sediment.

Keeping the trench open during the Bering Sea winter is the primary function of the sheet piling, because this work must be completed before the start of the first season of pipelay. Even though the actual pipe pull may be delayed until the period of the best weather is expected, the pull should not be further delayed because the trench work is not complete.

The area favored for bringing the pipe ashore lies in the vicinity of Port Moller, from which point the pipeline would be continued overland to the onshore terminal at Lefthand Bay.

The pipeline shore landing from the Bering Sea would probably be made on the middle barrier island of the Kudobin group, 1.6 nautical miles north of Cape Rozhnof. With the pulling winch located on this same island, the pipeline connection to Cape Rozhnof can be made by a pull across the mudflats of Nelson Lagoon. No laybarge is needed for this work. A pipeway would be built on Cape Rozhnof and pipe joints would be delivered to this point by road from Lefthand Bay.

From Cape Rozhnof to the southern end of Herendeen Bay, the pipeline can be continued by two long pulls directly across the bay. To do this, a laybarge would be anchored in Herendeen Bay 2 nautical miles west of Pt. Divide (see Figure 7 - 8). This is the termination of the Hague Channel, and the water depth here is greater than 15 fathoms - sufficient to float even a 3rd generation barge at working displacement. On either side of the Hague Channel, the water depth ranges from a shallow 3 fathoms to drying mudflats.

The two pipe pulls would be made without having to move the barge. The first pull would be made from the barge across Mud Bay to Cape Rozhnof (or vice versa), a distance of 5 nautical miles. The western arm of Deer Island lies across the route of the pull, and a trench below sea level would have to be excavated or dredged before the pull commences.

Once the pull to Cape Rozhnof is completed, the pipe would be buoyed off and set on the seabed. The barge would be turned around and the pull across Herendeen Bay to the vicinity of Buck Valley would be carried out. The distance is 8 nautical miles.

The pipeline water crossing is completed by recovering the pipe end from the Cape Rozhnof section on the barge's davits and joining it to the section terminated at Buck Valley.

The construction planning of these four pipe pulls from the Bering Sea to the southern end of Herendeen Bay will have to be developed in considerable detail to make the procedures cost effective and to minimize risk. Over long pulls, buoyancy may have to be added to the pipe to reduce friction forces, but any construction concepts involving complete flotation of the pipe should be viewed with caution in the Aleutian environment. Short chop produced in the bay under even typical wind conditions can exert impressive lateral forces on a floating pipeline.

In Section 7.7.4, emphasis was placed on the importance of advancing the pipeline construction schedule in the first season of pipelay. Further to this purpose, the pipe pulls in Herendeen

Bay should not be scheduled during the barges' working seasons at sea. Rather, they should be done prior to the first working season or between seasons when the laybarges are in winter layover status.

7.8 Engineering Considerations

Environmental data in the Bering Sea and onshore must be obtained to support detailed design of the pipeline and to establish the route. Offshore, the Bering Sea shelf is expected to be flat and free of obstacles, but this must be confirmed and the pipeline routed accordingly.

Data on seabed soil conditions would be obtained together with meteorological and oceanographic data pertinent to pipeline engineering.

The pipelines would be trenched nearshore - through the surf zone and out to a water depth great enough that the pipe can remain exposed in the seabed without danger of displacement from combined storm waves and currents. Twenty nautical miles of trenching was assumed to be sufficient, and a cost allowance is made for this in the pipeline construction estimate.

An onshore survey will also be made during the conceptual phase of the development to survey the route. These data will be incorporated into alignment sheets and route descriptions needed for scheduling and definitive cost estimation.

7.9 Summary and Conclusions

Pipelines for two cases of crude oil throughput have been described from each of the three lease basins to a terminal at Lefthand Bay. North Sea design and construction procedures have been postulated for the sea sections of the lines, and the lines are conservatively sized to transport oil of high viscosity. One 24-inch O.D. x 0.625 inch wall thickness pipeline will accommodate 100 MBPD and one 36-inch O.D. x 0.875 inch wall thickness will transport 300 MBPD.

It is anticipated that acceptable pipeline routes can be established from all three lease sale areas to a shore base on the Alaska Peninsula. No unusual obstacles or engineering obstacles are anticipated which are beyond the state of practice for other severe weather areas such as the North Sea.

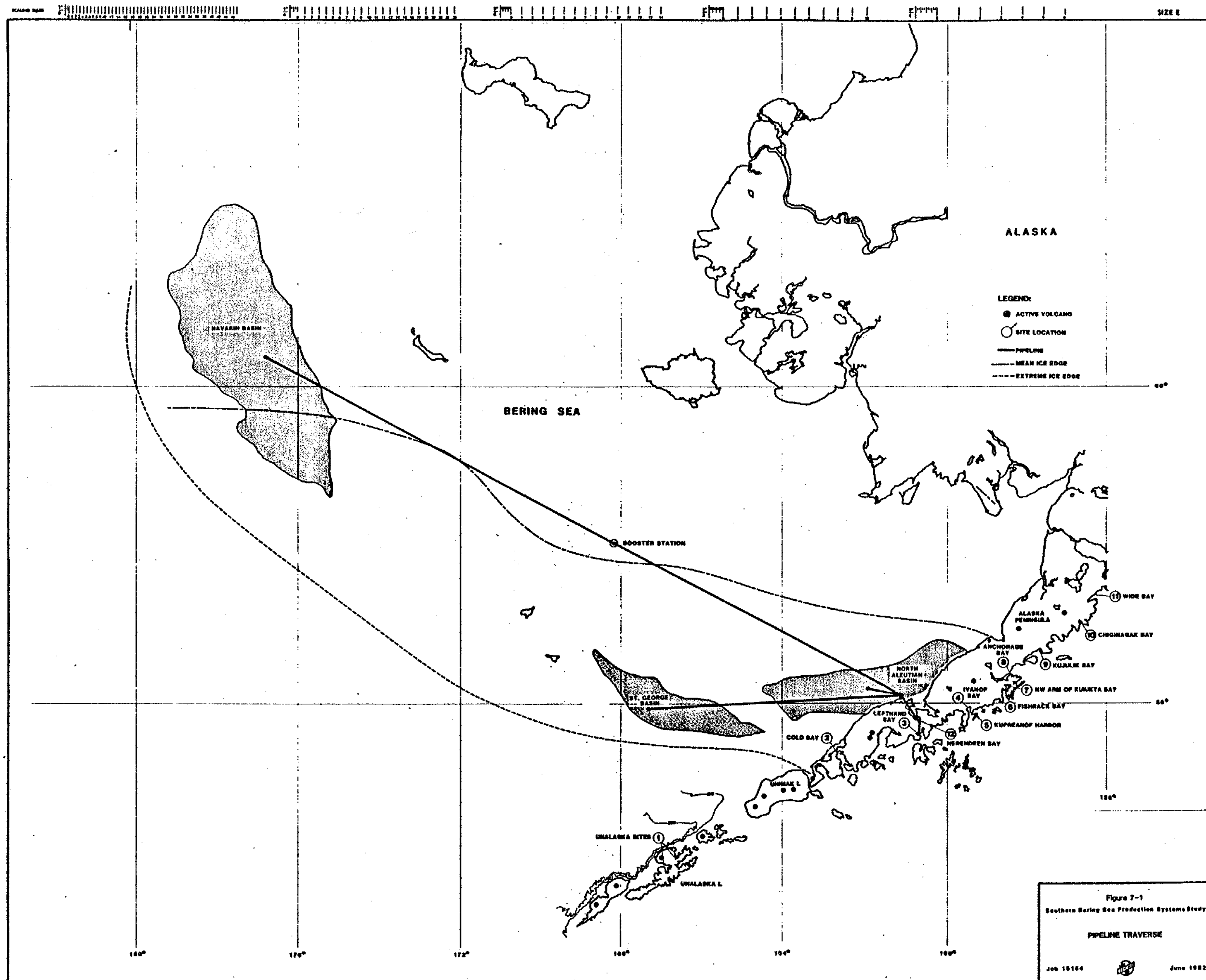
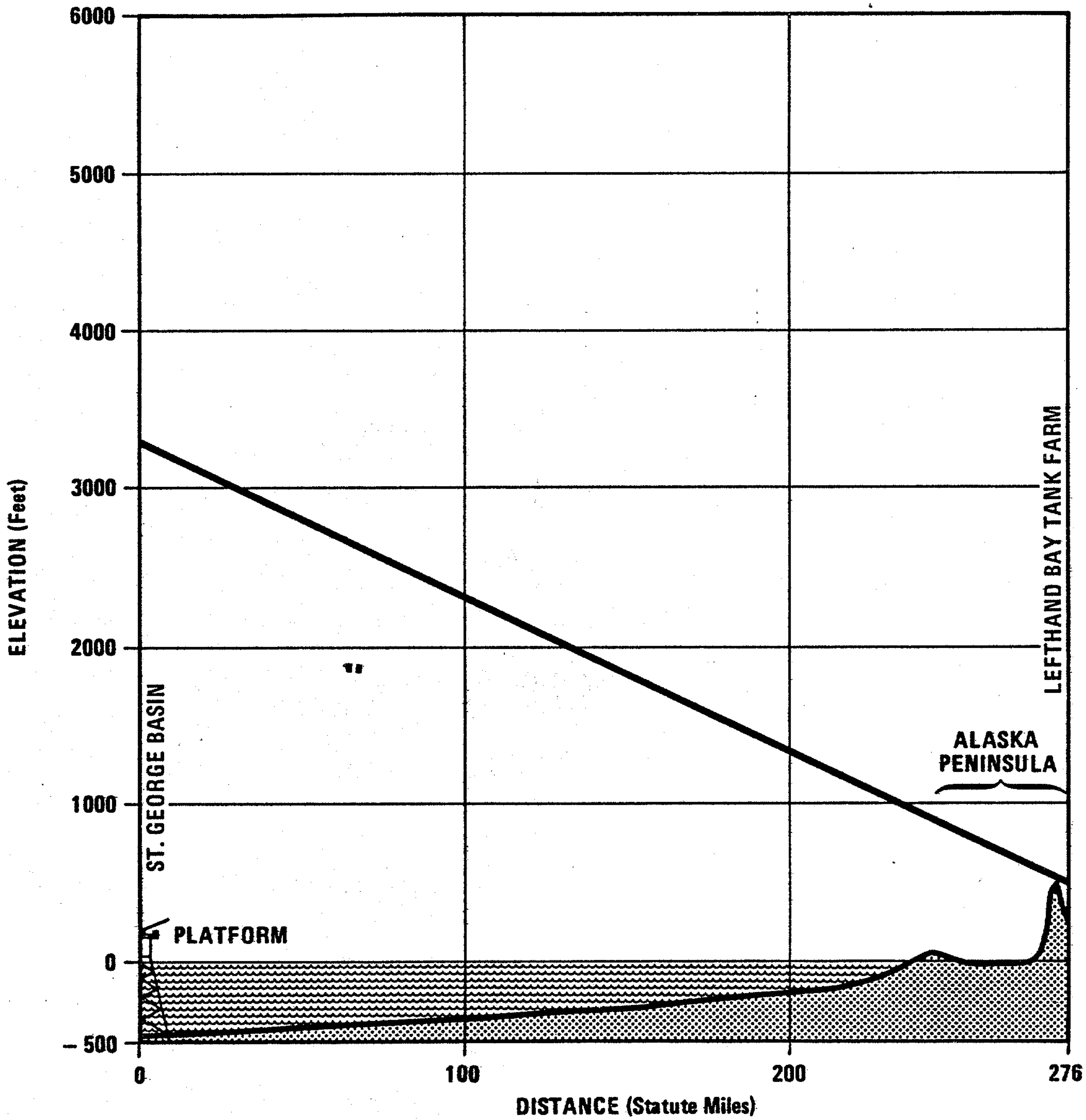


Figure 7-1
Southern Bering Sea Production Systems Study

PIPELINE TRAVERSE

Job 15164 June 1982



PIPE O. D.	24 INCHES
AMB. TEMP.	35°F
KIN. VISCOSITY	110 CENTISTOKES

Figure 7 – 2

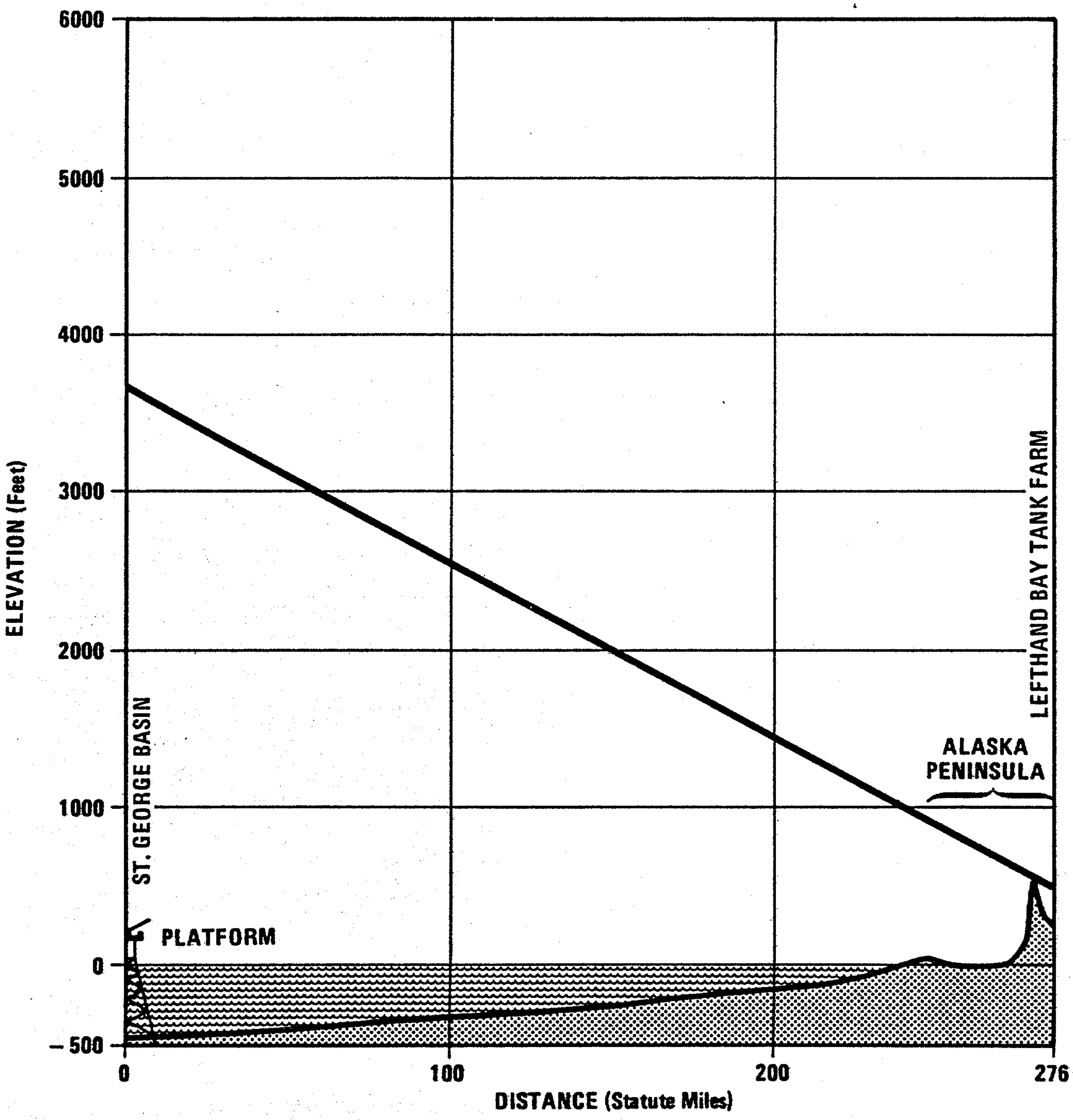
Southern Bering Sea Production Systems Study

St. George Basin To Lefthand Bay

24" Diameter – 100 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 BECHTEL June 1982



PIPE O. D. – 36 INCHES
AMB. TEMP. – 35°F
KIN. VISCOSITY – 450 CENTISTOKES

Figure 7-3
Southern Bering Sea Production Systems Study

St. George Basin To Lefthand Bay
36" Diameter – 300 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 BECHTEL June 1982

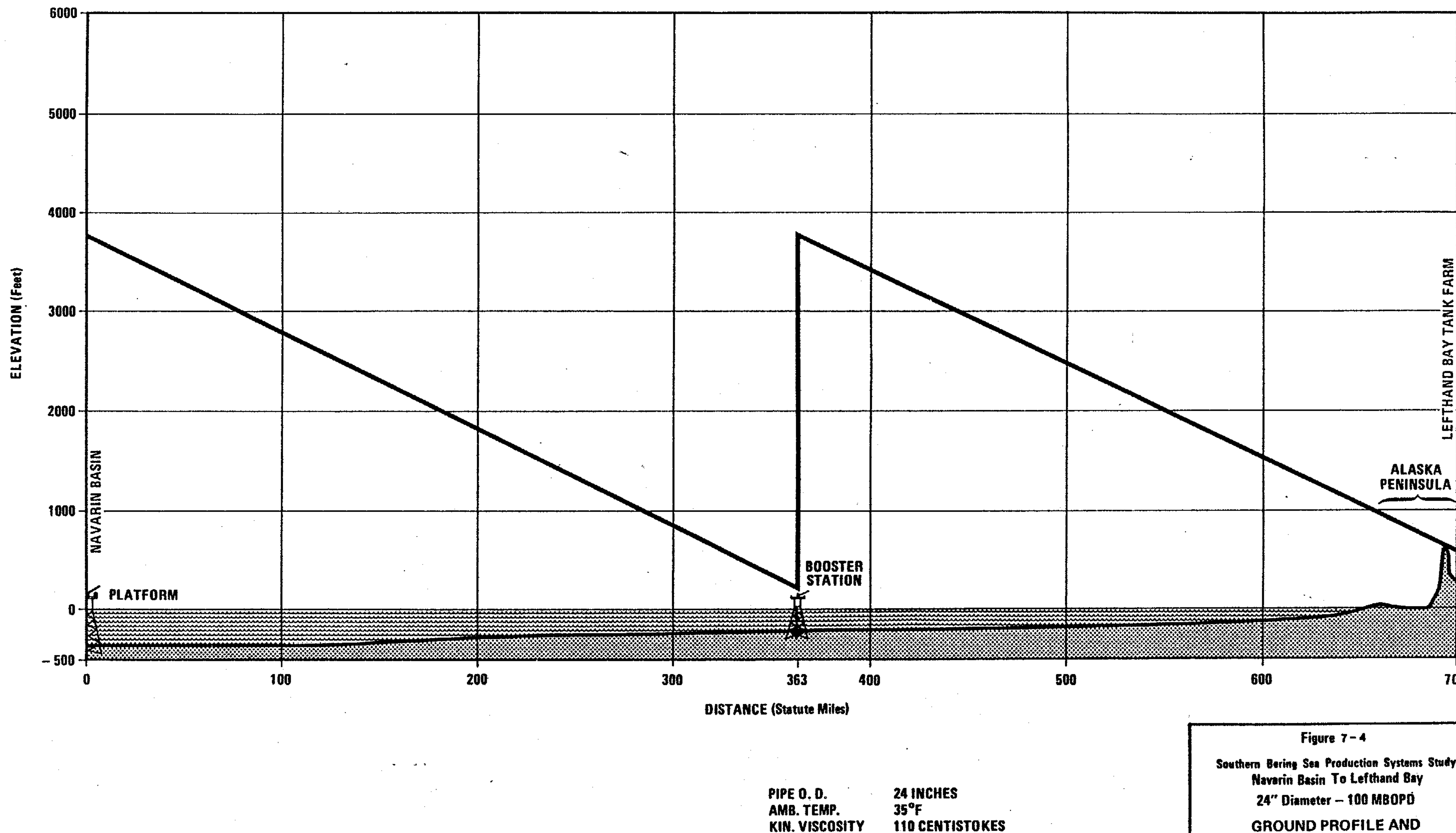
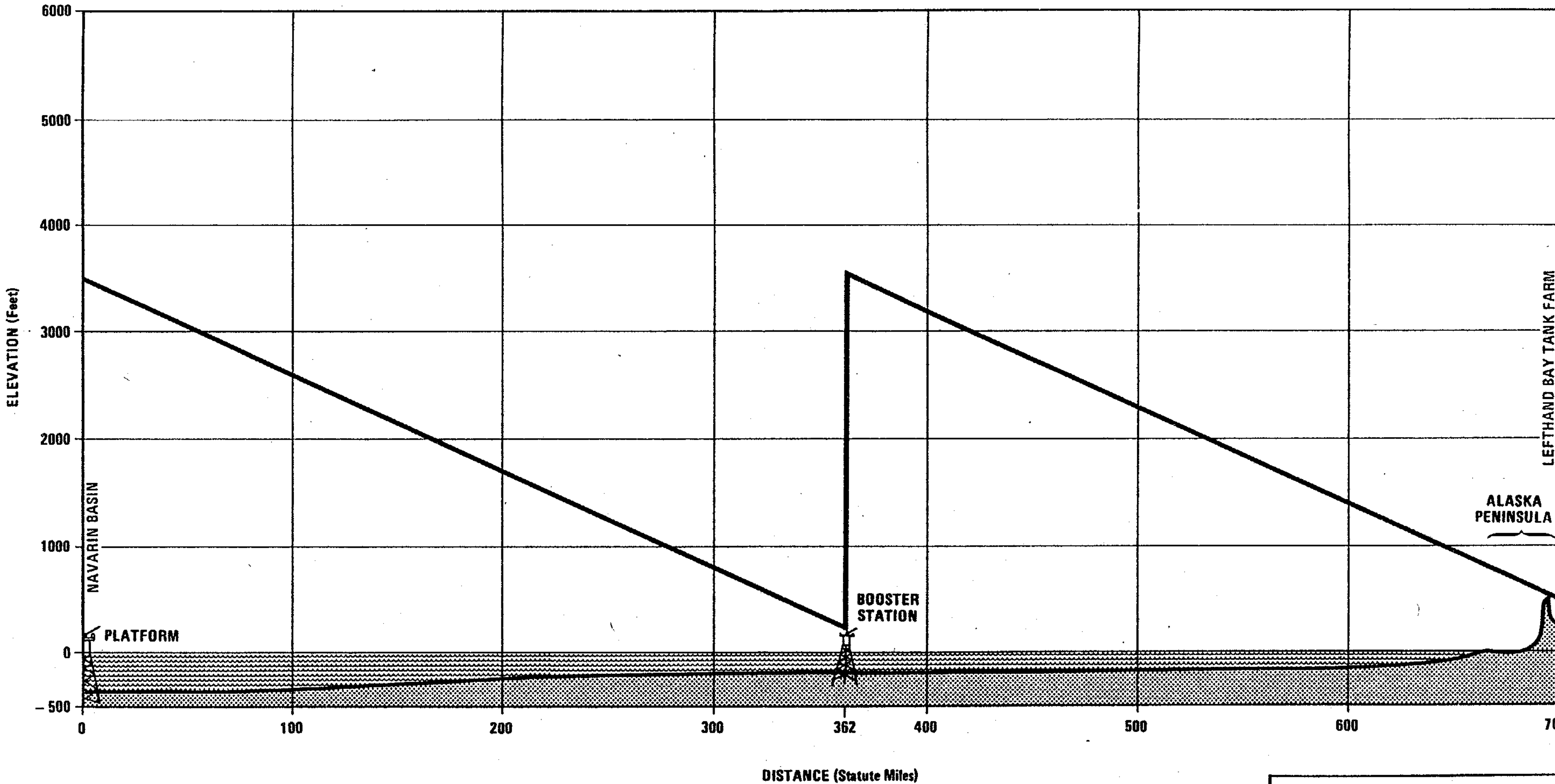


Figure 7 – 4

Southern Bering Sea Production Systems Study
Navarin Basin To Lefthand Bay
24" Diameter – 100 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 June 1982



PIPE O. D. 36 INCHES
AMB. TEMP. 35°F
KIN. VISCOSITY 110 CENTISTOKES

Figure 7 – 5
Southern Bering Sea Production Systems Study
Navarin Basin To Lefthand Bay
36" Diameter – 300 MBOPD

GROUND PROFILE AND HYDRAULIC GRADIENT

Job 15154 June 1982

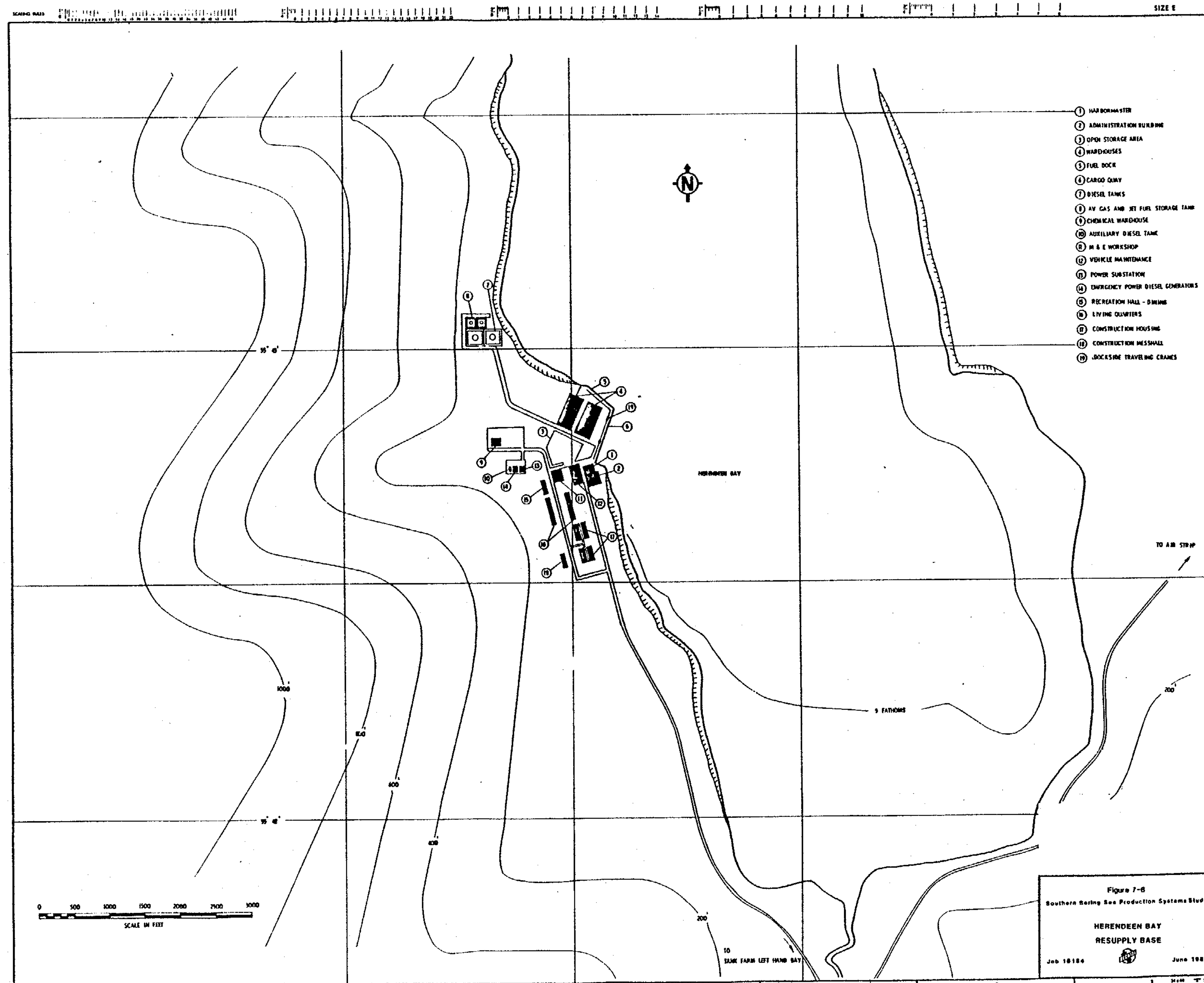


Figure 7-6
Southern Bering Sea Production Systems Study
HERENDEEN BAY
RESUPPLY BASE
Job 18184
June 1982

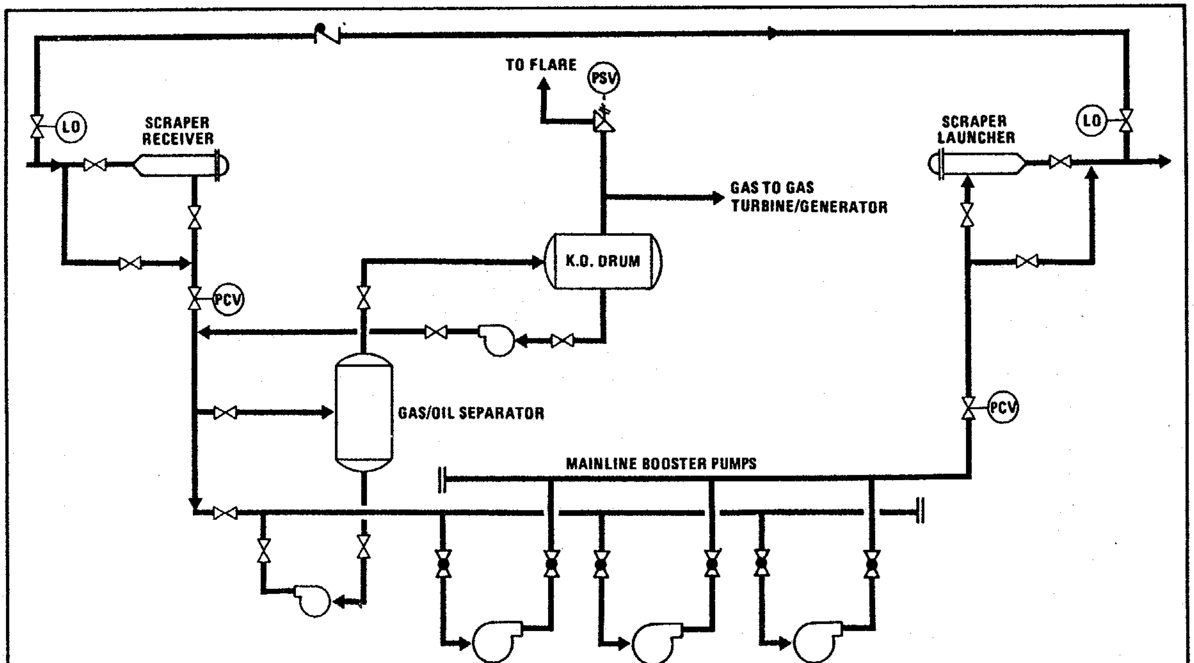


Figure 7-7

Southern Bering Sea Production Systems Study

BOOSTER STATION FLOW DIAGRAM

Job 15154 BECHTEL June 1982

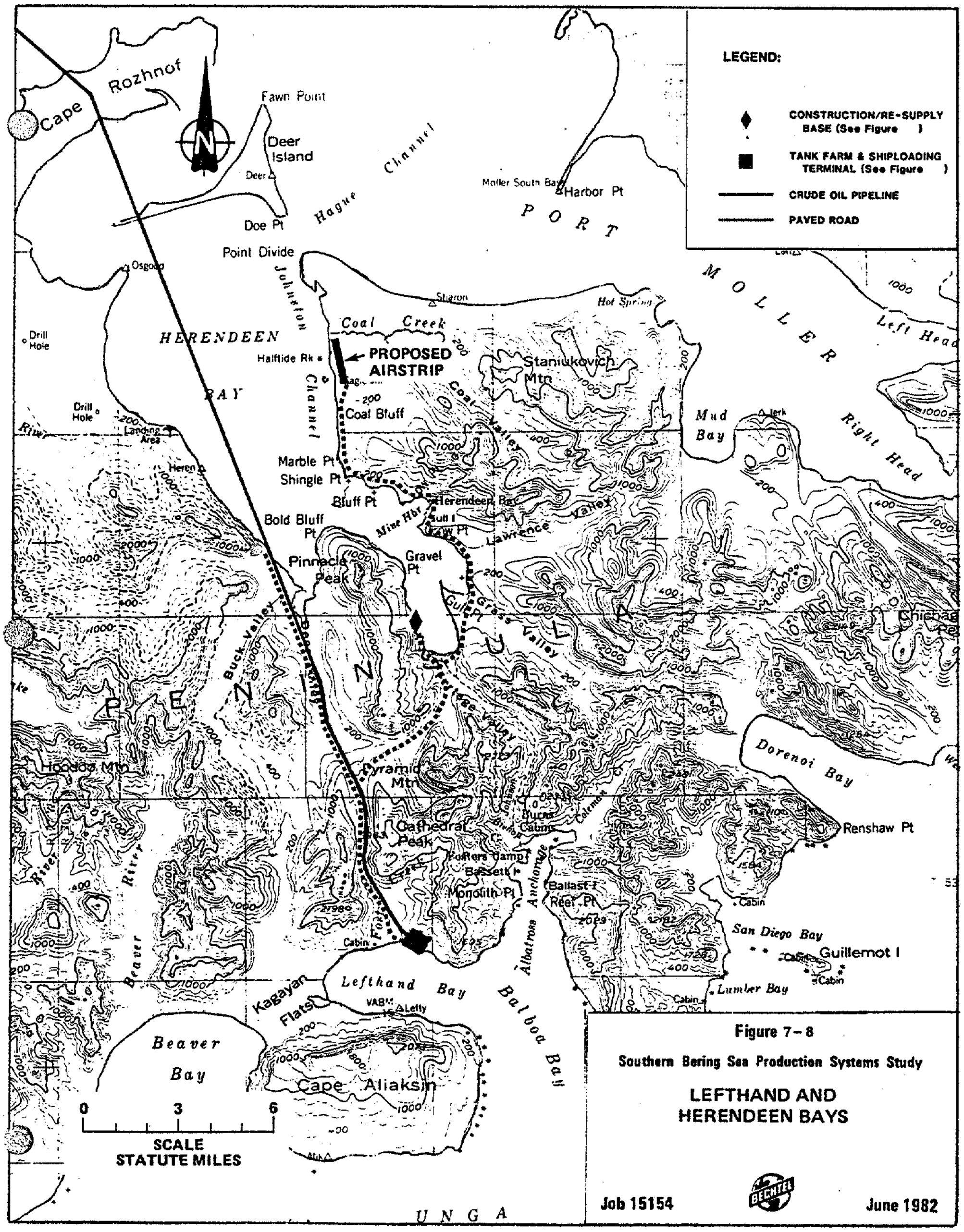


Figure 7-8

Southern Bering Sea Production Systems Study

LEFTHAND AND HERENDEEN BAYS

Job 15154 June 1982

TABLE OF CONTENTS

SECTION 8.0

8.0 SHORE TERMINAL

8.1 Scope of Work

A shore terminal for crude oil storage and tanker loading is an important alternative in the transportation system for Bering Sea production. The scope of work for this aspect of the study is to:

- Develop criteria for terminal site selection.
- Select favorable sites through an office study of charts, maps and other published data.
- Identify suitable terminal sites, preferably those best suited to service each of the three lease areas under consideration.
- Develop a layout of the terminal facilities for each of the three preferred sites and identify the major components of the facility.
- Select suitable sites for a production platform resupply base.
- Produce a layout of a typical base and identify the major components.
- Identify any engineering problems or constraints to development of the shore terminal.
- Develop order of magnitude costs for an onshore storage and tanker loading terminal suplied by a submarine pipeline from the offshore production platform. (Transshipment terminal locations were identified, but no layouts or cost estimates were prepared.)

8.2 Design Criteria and Assumptions

8.2.1 Shore Terminal Capacity

- o Crude storage - will accommodate at least 7 to 10 days of production for cases of 100 MBPD and 300 MBPD with capability of future expansion to 1000 MBPD.

- o Tanker size - tankers of up to 250,000 dwt can be accommodated at the loading berths.

8.2.2 Site Selection Criteria - the criteria chosen for evaluation of possible terminal sites are:

- o Protection of the berth from sea waves
- o Space for land facilities and future expansion
- o Ship berthing potential
- o Deepwater near shore
- o Turning basin
- o Submarine pipeline access
- o Land pipeline access from the point of shore landing of the submarine line
- o Tanker access to the berth
  - harbor approaches
  - offshore
- o Protection from wind at the berth
- o Proximity to offshore lease areas
- o Suitability as a tanker transshipment point (no pipeline to shore)
- o Suitability as an offshore construction or platform resupply base
- o Proximity of the terminal site to a separate resupply base site
- o Minimum ice cover

8.3 Site Selection

The three sedimentary basins in the southern Bering Sea considered for this study are the North Aleutian, St. George and Navarin Basins as shown in Figures 8-1 and 8-2 . The lease areas for each region extend beyond the bounds of the sedimentary basin; but, for simplicity, the likely location of a production platform is assumed to be at the center of each basin.

For the North Aleutian Basin, the nearest landfall for a submarine pipeline is the Alaska Peninsula. The St. George Basin is closest to the near Aleutian Islands and the Pribilof Islands. The northernmost basin, Navarin, is distant from all major land masses with the closest island being St. Matthew.

Besides having water depths capable of accommodating tankers in the assumed size range of 100,000 dwt to 250,000 dwt, a suitable port site should have protection against sea waves from all directions. The climate is severe in the Bering Sea area and in the Gulf of Alaska, and storm winds can come from any direction depending on the season of the year. Exposure of the loading berths to waves from any sector of the compass could result in an increase in port closure durations in some proportion to the extent of that exposure.

The Bering Sea Islands, St. Matthew and the Pribilofs, do not have natural harbor configurations in their shorelines that could shelter large ships. Anchorages can be found offshore of the islands which can provide a lee under certain weather conditions, but none of these can be upgraded to the status of an all-weather loading terminal.

The entire west coast of Alaska is low-lying with shallow offshore slopes. There are no harbors suited to admitting very large crude carriers, and winter sea ice along that coast would be an obstacle to tanker traffic.

Shallow coastal water continues to be found in Bristol Bay and along the north coast of the Alaska Peninsula, although the likelihood of seasonal sea ice occurrence decreases westwardly along this coast.

The south coast of the Alaska Peninsula, however, remains ice free throughout the year. In the worst conditions of sea icing experienced in recent history, during the winter of 1923-24, False Pass between the western tip of the Alaska Peninsula and Unimak Island was entirely blocked with ice. Drift ice from the pack in False Pass extended as far south as Sanak Island.* Even under these conditions, no sea ice formed on the south shores of the peninsula but rather invaded from the north side.

The current along the south side of the Alaska Peninsula, originating in the Gulf of Alaska, has been called a warm current. It causes the south side of the peninsula to be consistently warmer than the north or Bering Sea side.*

Proceeding westwards from the Alaska Peninsula into the near Aleutian Islands, the first likely deepwater ports are found which open on the Bering Sea. Three, which have some potential for development as tanker loading terminals, are all located on Unalaska Island. It is recommended that those sites be considered primarily as tanker transshipment terminals. As can be seen in Figure 8-1, very deep water below the edge of the Bering Sea Shelf is an obstacle to direct submarine pipeline routes from the St. George or Navarin Basins. Pipelines from these basins would be routed towards the Alaska Peninsula until the eastern edge of the shelf break is cleared and then along the Aleutian Ridge to Unalaska Island.

Possible island sites further west of Unalaska have little potential. The distance required to deliver crude oil to the onshore facility by pipeline or by tanker increases beyond Unalaska Island.

* United States Coast Pilot #9, Tenth Edition, January, 1981

There is reason to question whether a submarine pipeline route could be found from the principal basins even as far as Unalaska Island. The Bering Shelf ends at about the longitude of Unimak Pass. However, the seabed on the shelf is remarkably flat. A review of seabed data collected by NOAA survey vessels showed no presence of rock outcroppings. But a pipeline route from the Unimak Pass area to Unalaska Island, a distance of about 70 nautical miles, must follow the Aleutian Ridge inside the 600 foot depth contour. This is not because pipelines cannot be laid in deeper water, but because the north side of the Aleutian Ridge drops very rapidly into the Bering Canyon beyond the 100 fathom contour. The bathymetric contours on this continental slope show the seabed to be unfit for pipelay.*

In places, the 100-fathom contour comes within 200 feet of the island coasts. Elsewhere,the top of the Aleutian Ridge is marked with depressions, step-like seabed structures and rocky outcroppings. Whether the ridge top will permit a continuous, acceptable pipeline route to Unalaska Island and beyond can only be determined by a marine survey. For the purposes of this study, the existence of a suitable route is considered unlikely, and all land sites west of Unimak Pass are presumed to be only for tanker transshipment terminals or resupply bases.

Returning to the south side of the Alaska Peninsula, it can be seen in Figures 8-1 and 8-2 that the coast is generally irregular and mountainous. A number of active volcanos are distributed along the peninsula. Their locations are shown in Figure .

* "Bathymetry of the Aleutian Arc, Alaska, Plate 5," U.S. Dept. of Commerce Coast and Geodetic Survey, ESSA Monograph 3 - Nichols and Perry, 1966.

The frequency of occurrence of inlets, bays and offshore islands on the south coast of the peninsula suggest that deep water port sites could be abundant; and, in fact, 10 of the 14 sites identified lie in this area. Still more sites can be found further eastwards on the south shore of the peninsula and on the southern mainland of Alaska, but their distance from the lease basins removes them from consideration.

Few of these sites can give full protection against storms for lack of dominant directions in the storm winds. Many sites have major weaknesses in evaluations against the list of criteria given in Section 8.2, and all of these southern coast sites require an overland pipeline to the north shore.

In a comparison of all of the sites against the selection criteria (Figures 8-3 and 8-4), one emerges as clearly preferred provided there is to be a pipeline to shore. This is Site 3, Lefthand Bay.

The sites selected are described below -- not in order of their preference but from the westernmost eastwards.

Site 1 - Unalaska Island

Unalaska Island has three identified sites for tanker transshipment terminals (Figure 8-5). As discussed above, supply of crude oil to the terminal by submarine pipeline may not be possible. These sites could also serve as supply bases for offshore construction and platform resupply since they give good protection from the sea. In this regard, Dutch Harbor would also make a good base for supply boats, although it is too shallow to accommodate very large crude carriers.

TABLE OF CONTENTS

SECTION 8.0

LIST OF FIGURES

SECTION 8.0

8.0 SHORE TERMINAL

8.1 Scope of Work

A shore terminal for crude oil storage and tanker loading is an important alternative in the transportation system for Bering Sea production. The scope of work for this aspect of the study is to:

- Develop criteria for terminal site selection.
- Select favorable sites through an office study of charts, maps and other published data.
- Identify suitable terminal sites, preferably those best suited to service each of the three lease areas under consideration.
- Develop a layout of the terminal facilities for each of the three preferred sites and identify the major components of the facility.
- Select suitable sites for a production platform resupply base.
- Produce a layout of a typical base and identify the major components.
- Identify any engineering problems or constraints to development of the shore terminal.
- Develop order of magnitude costs for an onshore storage and tanker loading terminal suplied by a submarine pipeline from the offshore production platform. (Transshipment terminal locations were identified, but no layouts or cost estimates were prepared.)

8.2 Design Criteria and Assumptions

8.2.1 Shore Terminal Capacity

- o Crude storage - will accommodate at least 7 to 10 days of production for cases of 100 MBPD and 300 MBPD with capability of future expansion to 1000 MBPD.

- o Tanker size - tankers of up to 250,000 dwt can be accommodated at the loading berths.

8.2.2 Site Selection Criteria - the criteria chosen for evaluation of possible terminal sites are:

- o Protection of the berth from sea waves
- o Space for land facilities and future expansion
- o Ship berthing potential
- o Deepwater near shore
- o Turning basin
- o Submarine pipeline access
- o Land pipeline access from the point of shore landing of the submarine line
- o Tanker access to the berth
  - harbor approaches
  - offshore
- o Protection from wind at the berth
- o Proximity to offshore lease areas
- o Suitability as a tanker transshipment point (no pipeline to shore)
- o Suitability as an offshore construction or platform resupply base
- o Proximity of the terminal site to a separate resupply base site
- o Minimum ice cover

8.3 Site Selection

The three sedimentary basins in the southern Bering Sea considered for this study are the North Aleutian, St. George and Navarin Basins as shown in Figures 8-1 and 8-2 . The lease areas for each region extend beyond the bounds of the sedimentary basin; but, for simplicity, the likely location of a production platform is assumed to be at the center of each basin.

For the North Aleutian Basin, the nearest landfall for a submarine pipeline is the Alaska Peninsula. The St. George Basin is closest to the near Aleutian Islands and the Pribilof Islands. The northernmost basin, Navarin, is distant from all major land masses with the closest island being St. Matthew.

Besides having water depths capable of accommodating tankers in the assumed size range of 100,000 dwt to 250,000 dwt, a suitable port site should have protection against sea waves from all directions. The climate is severe in the Bering Sea area and in the Gulf of Alaska, and storm winds can come from any direction depending on the season of the year. Exposure of the loading berths to waves from any sector of the compass could result in an increase in port closure durations in some proportion to the extent of that exposure.

The Bering Sea Islands, St. Matthew and the Pribilofs, do not have natural harbor configurations in their shorelines that could shelter large ships. Anchorages can be found offshore of the islands which can provide a lee under certain weather conditions, but none of these can be upgraded to the status of an all-weather loading terminal.

The entire west coast of Alaska is low-lying with shallow offshore slopes. There are no harbors suited to admitting very large crude carriers, and winter sea ice along that coast would be an obstacle to tanker traffic.

Shallow coastal water continues to be found in Bristol Bay and along the north coast of the Alaska Peninsula, although the likelihood of seasonal sea ice occurrence decreases westwardly along this coast.

The south coast of the Alaska Peninsula, however, remains ice free throughout the year. In the worst conditions of sea icing experienced in recent history, during the winter of 1923-24, False Pass between the western tip of the Alaska Peninsula and Unimak Island was entirely blocked with ice. Drift ice from the pack in False Pass extended as far south as Sanak Island.* Even under these conditions, no sea ice formed on the south shores of the peninsula but rather invaded from the north side.

The current along the south side of the Alaska Peninsula, originating in the Gulf of Alaska, has been called a warm current. It causes the south side of the peninsula to be consistently warmer than the north or Bering Sea side.*

Proceeding westwards from the Alaska Peninsula into the near Aleutian Islands, the first likely deepwater ports are found which open on the Bering Sea. Three, which have some potential for development as tanker loading terminals, are all located on Unalaska Island. It is recommended that those sites be considered primarily as tanker transshipment terminals. As can be seen in Figure 8-1 , very deep water below the edge of the Bering Sea Shelf is an obstacle to direct submarine pipeline routes from the St. George or Navarin Basins. Pipelines from these basins would be routed towards the Alaska Peninsula until the eastern edge of the shelf break is cleared and then along the Aleutian Ridge to Unalaska Island.

Possible island sites further west of Unalaska have little potential. The distance required to deliver crude oil to the onshore facility by pipeline or by tanker increases beyond Unalaska Island.

* United States Coast Pilot #9, Tenth Edition, January, 1981

There is reason to question whether a submarine pipeline route could be found from the principal basins even as far as Unalaska Island. The Bering Shelf ends at about the longitude of Unimak Pass. However, the seabed on the shelf is remarkably flat. A review of seabed data collected by NOAA survey vessels showed no presence of rock outcroppings. But a pipeline route from the Unimak Pass area to Unalaska Island, a distance of about 70 nautical miles, must follow the Aleutian Ridge inside the 600 foot depth contour. This is not because pipelines cannot be laid in deeper water, but because the north side of the Aleutian Ridge drops very rapidly into the Bering Canyon beyond the 100 fathom contour. The bathymetric contours on this continental slope show the seabed to be unfit for pipelay.*

In places, the 100-fathom contour comes within 200 feet of the island coasts. Elsewhere,the top of the Aleutian Ridge is marked with depressions, step-like seabed structures and rocky outcroppings. Whether the ridge top will permit a continuous, acceptable pipeline route to Unalaska Island and beyond can only be determined by a marine survey. For the purposes of this study, the existence of a suitable route is considered unlikely, and all land sites west of Unimak Pass are presumed to be only for tanker transshipment terminals or resupply bases.

Returning to the south side of the Alaska Peninsula, it can be seen in Figures 8-1 and 8-2 that the coast is generally irregular and mountainous. A number of active volcanos are distributed along the peninsula. Their locations are shown in Figure .

* "Bathymetry of the Aleutian Arc, Alaska, Plate 5," U.S. Dept. of Commerce Coast and Geodetic Survey, ESSA Monograph 3 - Nichols and Perry, 1966.

The frequency of occurrence of inlets, bays and offshore islands on the south coast of the peninsula suggest that deep water port sites could be abundant; and, in fact, 10 of the 14 sites identified lie in this area. Still more sites can be found further eastwards on the south shore of the peninsula and on the southern mainland of Alaska, but their distance from the lease basins removes them from consideration.

Few of these sites can give full protection against storms for lack of dominant directions in the storm winds. Many sites have major weaknesses in evaluations against the list of criteria given in Section 8.2, and all of these southern coast sites require an overland pipeline to the north shore.

In a comparison of all of the sites against the selection criteria (Figures 8-3 and 8-4), one emerges as clearly preferred provided there is to be a pipeline to shore. This is Site 3, Lefthand Bay.

The sites selected are described below -- not in order of their preference but from the westernmost eastwards.

Site 1 - Unalaska Island

Unalaska Island has three identified sites for tanker transshipment terminals (Figure 8-5). As discussed above, supply of crude oil to the terminal by submarine pipeline may not be possible. These sites could also serve as supply bases for offshore construction and platform resupply since they give good protection from the sea. In this regard, Dutch Harbor would also make a good base for supply boats, although it is too shallow to accommodate very large crude carriers.

The three sites are Wide Bay at the northwest side of Unalaska Bay, Captains Bay northwest of Dutch Harbor, and Anderson Bay, the southern arm of Makushin Bay.

Wide Bay allows for tanker berthing close to shore, but little space is available on land for terminal facilities.

Captains Bay is favorably located near Dutch Harbor and the existing airstrip, but the entrance to the bay is narrow, less than 600 ft. wide. Plans for developing this site should give consideration to widening the entrance or possibly locating berths outside at the southern end of Unalaska Bay (west of Amaknak Island on which Dutch Harbor airport is located).

Anderson Bay affords good, deepwater anchorage near shore on the west side. Space for onshore facilities is limited. This site is the closest of the three to Makushin Volcano which had a minor explosive eruption as recently as 1938 and a major eruption in 1826.*

Site 2 - Cold Bay (162° 39'W, 55° 12'N)

Cold Bay is a large bay almost spanning the Alaska Peninsula (Figure 8-6). There is a small community on the west side of the bay and an airstrip at the abandoned Thornbrough Air Force Base which is used by commercial aircraft.

Deep water for large tankers lies close to shore south of the village. Flat, open land adjacent to the tanker berths is plentiful for tank farms and facilities.

* Coats, Robert R., "Volcanic Activity in the Aleutian Arc," Geological Survey Bulletin 974-B, Contributions to General Geology, 1950.

This site is closest to the offshore lease basins in pipeline distance among all sites selected.

Although the anchorage in Cold Bay is well protected from sea waves, the flat land surrounding it provides no protection against wind from the northerly sector. Average monthly wind speeds of 20 miles per hour and more are recorded at Cold Bay; and, during winter months, visibilities are frequently restricted because of the blowing snow.*

The major disadvantage of Cold Bay as a tanker loading terminal site concerns the navigational approaches to the bay itself. Within the bay, the water depth is adequate to accommodate 250,000 dwt tankers; but the bay must be entered through a shallow, narrow channel. This channel is 3 nautical miles in length and runs due north close by Kelp and Kaslokan Points on the eastern shore. The channel is naturally maintained by tidal currents.

A great number of bathymetric soundings have been made in this channel by U.S. Government survey teams. From these data, cross sections through the channel have been drawn. It is found that the deep section of the channel has a minimum depth of 10 fathoms over a mean channel width of 750 feet. The eastern bank of the channel is steep and apparently rocky. Over a distance of about 1.5 nautical miles between Kelp and Kaslokan Points, both sides of the channel are hard, for the channel itself is a low point in a reef that extends almost across the entrance to Cold Bay.

The channel would have to be widened and deepened to admit tankers safely. Tugs and pilotage would be required. Delays can be expected when sea waves or wind make channel transit hazardous.

* Coast Pilot #9

The principal concern regarding the navigational approaches to Cold Bay is not the entrance channel but rather the shoal areas in the only likely southern approach, Deer Passage between Deer Island and the peninsula mainland. The water depth between this passage and the entrance to Cold Bay deserves further survey, even a sweep, before Cold Bay is accepted as a suitable terminal for very large crude carriers.

Site 3 - Lefthand Bay of Balboa Bay (160° 43'W, 55° 33'N)

Lefthand Bay lies on the west side of Balboa Bay and is known locally as Left Arm* (Figure 8-7).

Over a distance of about 15 miles through a low pass (200 feet elevation), access can be gained to Herendeen Bay on the north side of the Alaska Peninsula. Although Herendeen Bay is unsuitable as a tanker terminal for lack of sufficient water depth, it is well suited as a site for a construction/resupply base (Section 8.6).

As shown in Figures 8-3 and 8-4 , Lefthand Bay is the preferred site among those reviewed in the categories of tanker berthing and onshore development potential. There is deep water close to shore, adequate shore front for berth expansion and complete protection from sea waves. From Figure 8-7 , it would appear that the terminal site is exposed to waves from the southeast. However, Popof and Korovin Islands lie less than 10 nautical miles southeast of the harbor entrance. With a fetch of less than 10 miles, wind-generated waves have a short period. The shorter the wave period, the more the wave height is degraded by the diffraction required to propagate these waves from Balboa Bay into the selected terminal site.

* Coast Pilot # 9, p. 203

Even severe southeast wind conditions are not likely to close the tanker berths, although such conditions might close the loading dock (Figure 8-15). Should this be the case, the smallcraft, tugs and launches, would find shelter at the head of Lefthand Bay. Further investigation into the frequency of occurrence of strong southwest wind conditions may indicate the desirability of permanent docking facilities for these smallcraft at the head of the bay.

Williwaws, the gusty winds funneled by the mountainous terrain of the south side of the Alaska Peninsula, may occur occasionally at Lefthand Bay, but no explicit note of this is taken in the Coast Pilot.

Space is available for the onshore facilities near the site selected for the ship berths. As seen in the layout of the 300,000 BPD case (Figure 8-15), an area for further expansion lies to the east of the initial development.

The nearest volcano to the Lefthand Bay site is Mount Dana about 25 miles to the west. This volcano does not appear to have been active in recent times.

Pavlof Volcano, which lies 50 miles southwest, is the closest active volcano having shown various levels of activity as recently as 1942 through 1948.

Site 4 - Ivanof Bay (159° 29'W, 55° 54'N)

Ivanof Bay (Figure 8-8) affords good, sheltered anchorage and short, shore approaches for tanker loading trestles at the head of the bay.

Road Island, situated in the center of the bay, limits the width of the approach channel into the head of the bay to half a nautical mile. This is wide enough to permit tanker transit without tug assistance. The island performs the important function of intercepting waves from the south, for it is stated that the north part of Ivanof Bay is well protected from southern swell.*

A cannery presently occupies the site selected for the onshore facilities, and its service wharf lies about halfway between the loading dock and tanker trestle shown for the facilities layout in Figure 8-16 . The wharf has a least depth of 22 feet alongside. This wharf would be useful in the initial stages of the onshore development.

An abundance of space is available for onshore expansion towards the northeast.

The land pipeline from the north coast of the peninsula is shown in Figure 8-8 . It is not only long, but a third of it must be constructed through mountainous terrain. Mount Veniaminof lies due north of the port site causing the pipeline route to deviate westwards. This volcano erupted explosively in 1892, emitted smoke in 1930 and produced a series of minor explosive eruptions in 1939. It underwent some unspecified activity in 1944. This volcano is only 15 miles from the site of the facilities, and its potential for future activity might be a factor in considering Ivanof Bay for a tanker loading terminal.

<u>Site 5 - Kupreanof Harbor (159° 21'W, 55° 47'N)</u>

Kupreanof Harbor lies at the west entrance to the strait between Jacob Island and Paul Island east of the Kupreanof Peninsula (Figure 8-8). Adequate water depth for tankers exists only at this western entrance to the strait, so the tanker approach must be from the west passing to the south of Ivanof Bay.

* Coast Pilot #9

Land area for onshore facilities is available on Paul Island, and trestle structures for tanker loading could be located at the tip of the hook on the western edge of the island. The structure of the islands and the confined sea passages among them suggest the potential for locally strong and irregular tidal currents. It is reported that the current movement within Kupreanof Harbor is indeed irregular in both direction and speed with velocities of one knot having been observed.

Kupreanof Harbor has the disadvantage as a site for onshore storage facilities of requiring a submarine pipeline connection to the Alaska Peninsula. The pipeline would originate somewhere near Alexander Point with a crossing distance of 1.7 statute miles. If the seabed is suitable, the pipeline construction can be done with a pipe pull from one or the other shores. The maximum water depth in the crossing is about 230 feet.

The on-land approach of the pipeline to Alexander Point passes through about 8 miles of very rugged terrain on the peninsula which comprises the eastern shore of Ivanof Bay. If Kupreanof Harbor is taken under further review as a port site, consideration should be given to the alternative of a submarine pipeline from the head of Ivanof Bay directly into the harbor. This distance is about 10 miles.

Site 6 - Fishrack Bay (158° 46'W, 56° 01'N)

Fishrack Bay is a branch of Mitrofania Bay as shown in Figure 8-9 . The northwest arm of Fishrack Bay has the necessary water depth and protection from the sea to function as a suitable terminal site. However, there is little space for onshore development. About a one-half square mile of flat land adjoins the shore of the bay, and this area is surrounded by hills rising steeply to over 1000 feet above sea level. Judging from the terminal facility layouts developed for three of the sites (Figures 8-14 through 8-16), it seems possible to build tankage and loading facilities for up to 300 MBPD on this land area. But there would be no room for further expanding the facilities.

A better approach to development of this site would be the construction of a tunnel through a ridge to the head of Ivan Bay, a distance of 7500 feet. The tunnel should have a large enough bore to permit not only vehicular traffic, but also to contain piping and electrical service to Fishrack Bay where the berths would be located. The main terminal facilities would be situated on the flood plain adjoining Ivan Bay where there exists all of the space necessary for future expansion.

<u>Site 7 - Portage Bay, the Northwest Arm of Kuiukta Bay (158° 36'W, 55° 11'N) and Site 8 - Anchorage Bay (158° 23'W, 55° 18'N)</u>

These two sites are treated together, because both may be required in the long-term development of a crude oil terminal complex.

Anchorage Bay is a deep inlet in the south shore of Chignik Bay about 3/4 of a nautical mile wide at its entrance by 1 nautical mile in length. The bay provides excellent protection along its east shore from waves, but it is noted in the Coast Pilot that heavy winds and williwaws are prevalent here.

An abandoned cannery and the village of Chignik are located on the east side of the bay where the tanker trestles would be most favorably situated. The ground rises steeply behind the village and herein lies the principal disadvantage of the site -- no space for growth of facilities.

A strip of gently sloping land parallels the east shore northwards from the village for about 1 mile with a maximum width of 1200 feet. A similar but smaller strip of land adjoins the southwest shore of the bay, and the rest of the shoreline rises steeply from the water.

Onshore storage and support facilities could conceivably be constructed at Anchorage Bay for up to 100 MBPD throughput, but another site is needed for handling increased production.

The overland pipeline from the Bering Sea coast would approach Anchorage Bay along the shore of Chignik Lake, parallel the Chignik River and divert towards the hills surrounding Anchorage Bay along the south shore of Chignik Lagoon. At about the location of the Metrofania Creek crossing a branch off the main pipeline can be constructed over flat ground southwards to the head of Portage Bay. There, expansion facilities can be constructed for crude storage and tanker loading.

Further investigation of the Portage Bay site may show it to be favored over Anchorage Bay; in which case, even the initial development would be situated here. At this time, several reservations regarding the value of Portage Bay as a terminal for very large crude carriers preclude a recommendation to this effect.

- Although "the water is generally deep close to shore throughout Kuiukta Bay and with few known exceptions in the arms leading from it"* there is no published bathymetry for Kuiukta Bay. Detailed bathymetric survey of the bay may show it not to be navigable for the whole range of tanker sizes assumed for this study.

* Coast Pilot #9, p. 194.

- It is remarked in the Coast Pilot that Kuiukta Bay is a natural funnel for winds, and it is known as one of the windiest bays in Alaska. It is further noted that, during a 36 hour period of west and northwest winds, almost continuous fresh winds and williwaws accompanied by fog and mist were experienced in Windy Bay. Portage Bay, too, is exposed to Northwest winds funneling through the low valley from Chignik Lagoon. As one can see in Figure , Windy Bay and Portage Bay are connected by a low valley on the route of Spoon Creek. The weather conditions reported above for Windy Bay must certainly have first passed through the proposed berthing area in Portage Bay.

- Kuiukta Bay extends more than 14 miles inland to the berthing area, and the presumed ship channel through the bay is narrowed in places by the occurrence of islands. Tanker assist by tugs may be not only advisable but mandatory, especially when the tanker is at ballast draft.

During periods of gustiness, Kuiukta Bay may be closed to tanker traffic.

Site 9 - Kujulik Bay (157° 45'W, 56° 40'N)

The tanker berthing area is located at the north end of Kujulik Bay (Figure 8-10) where deep water can be found close to shore. Unlike the other sites selected, this one is not fully protected from sea waves. It is exposed to waves from the south.

The entrance window for waves entering the bay is narrowed by a shallow reef extending southwest from Cape Kumlik which bares at mean lower low water. Significant reduction in waveheights can be expected at the berthing area due to diffraction of waves entering between the reef structure and Cape Kumliun. Neverthele s, offshore loading through single-point moorings is more practical for this site rather than trestle structures. Several such moorings can be situated at the north end of the bay without producing crowded traffic conditions.

Sufficient onshore space is available for facilities development as can be seen in the figure.

Easy onshore pipeline access can be found from this site into the Meshik River basin and from there northwestwards to the vicinity of Port Heiden on the Bering Sea.

Aniakchak Volcano lies 20 miles to the north-northwest. This volcano erupted explosively with some lava flow on May 20, 1931.

<u>Site 10 - Chiginagak Bay (156° 43'W, 57° 01'N)</u>

Chiginagak Bay owes its value as a tanker berthing area to Derickson Island in the center of the bay. This island produces a lee in the inner bay against south and southeast weather (Figure 8-11). How well it performs in this capacity is not clear from the published literature - most particularly the Coast Pilot #9. The extensive shallow region extending into the inner bay from behind Derickson Island suggests that wave systems would be channeled or focussed into the head of the bay with little opportunity for attenuation through diffractive processes.

The Coast Pilot indirectly confirms this (p. 188) ... "In S weather, better shelter can be found 500 yards N of Derickson Island in 13 fathoms. In moderate S weather very little swell makes into the anchorages." In essence, this advice indicates that the shorter period swell associated with moderate seas from the south find the water deep enough behind Derickson Island to diffract and lose waveheight.

As the description of the sea goes from moderate to heavy, the peak of the energy spectrum moves towards the longer periods. These longer waves "feel bottom" over the whole depth of the bay behind Derickson Island. These undergo no diffraction and only little refraction behind Derickson Island - enough, perhaps to develop a minor caustic beyond 500 yards behind the island.

It is not practical to locate tanker berths in this point of shelter, but up to two single-point, offshore tanker berths could be accommodated within the whole inner bay. If expansion plans require more than two berths or more than about 300 MPBD of throughput, Chiginagak Bay is not a suitable candidate for the siting.

Site 11 - Wide Bay (156° 25'W, 57° 22'N)

Wide Bay is a fine, well-protected, deep-water harbor with only a short land pipeline run to the shores of Bristol Bay. Deep water approaches close to shore in the area designated for tanker loading trestles (Figure 8-12).

An extensive area is available for onshore construction.

An airstrip or landing area exists at Short Creek, another at the village of Ugashik enroute to the north coast and still a third at Pilot Point on Ugashik Bay.

Well-suited though it is to harboring tankers, Wide Bay is not entirely free of problems. Here, the concern is with the entrance channels to the bay of which there are only two among the islands and reefs which protect the bay from the southeast. The two are separated by Channel Rock and are bounded by West Channel Island and East Channel Island.

The Coast Pilot recommends using the east channel, but it is the west channel that should be developed for tanker use. It is presently the deeper of the two. However, both channels are too narrow, each being about 1200 feet wide. Blasting and rubble removal over a distance of 3500 feet will be required to widen the west channel.

Once inside the bay at the head of the channel, the tankers will encounter a sufficiently broad expanse of deep water to make the 100° turn towards the berthing area. Some dredging in soft materials may be needed northwest of Terrace Island to establish suitable depths for the largest tankers, but from Hartman Island to the berths, water depths are on the order of 20 to 30 fathoms.

Site 12 - Herendeen Bay (160° 42'W, 54° 43'N)

Herendeen Bay on the north coast of the Alaska Peninsula has been selected for purposes of siting a construction/resupply base. A base here would be most appropriate in support of a tanker terminal facility at Site 3, Lefthand Bay. For this terminal location, Herendeen Bay will also be used for construction of the pipeline from its point of landing on the Bering Sea to a shore landing on the south side of the bay. As shown in Figure , it is proposed to pull the line across the entire bay. This procedure is discussed in Section 7.7.6.

The entrance to the bay lies opposite the town of Port Moller. Access is gained first by the Hague Channel which runs southwest from Port Moller, passing between Deer Island and Point Divide, to within 2 nautical miles of Black Point. Below Deer Island, the depth of the Hague Channel increases to between 15 and 25 fathoms. This depth will figure in the proposed pipeline construction procedures. Elsewhere, the channel depth lies between 5 and 7 fathoms.

: of Point Divide, the Johnston Channel branches off the
:nnel and extends southwards about 7 miles to a deep basin
:ad of Herendeen Bay where the supply base will be
The Johnston Channel with a least depth of 5 fathoms is
ı places but it is reasonably straight. Properly marked
;ation under foggy conditions, it will be negotiated
supply boats, all of which should be equipped with bow
.

channels through mudflats which are caused and maintained
currents, these channels are subject to meandering.
a note on Navigational Chart 16363, "Port Moller and
Bay" to this effect stating "Channels are subject to
changes due to very strong tidal currents." It would be
o maintain a buoy - handling survey boat for frequent
g of Johnston Channel, particularly during the early
development in Herendeen Bay. As operational
ty develops, the supply-base harbor master may reduce the
of channel verifications.

and supplies to be transported by supply boat into the
a will be delivered at a cargo dock in Lefthand Bay or by
e airstrip on Point Divide (Figure 8-7). These will
orted overland by truck to the supply base.

rship on the Alaska Peninsula

-13 shows land ownership and the distribution of state
al jurisdictions. Any small, private holdings would not
a map of this small a scale.

s presumably recent, but its date of issue is not known.
e is the U.S. Fish and Wildlife Service, Bristol Bay

Cooperative Management Plan, in conjunction with the Alaska Department of Natural Resources.

8.5 Terminal Layout and Design

Terminal layouts have been produced fo three sites, Cold Bay, Lefthand Bay and Ivanof Bay (Figures 8-14 , 8-15 and 8-16). They all have the same components for a 300 MBPD facility, but they differ in arrangement according to the demands of the topography and tanker mooring areas.

These three sites were selected for layout because they are the closest on the Alaska Peninsula to the offshore production sites. Should circumstances require the actual development of a site further east, the penalty would be additional length of pipeline for transportation of crude ashore. The favored terminal site is Lefthand Bay. Descriptions of the facilities will be restricted to this site (Figure 8-15). The primary components are called out in the legend on the figure.

Crude oil with some natural gas liquids is delivered by pipeline to the process area. Here the crude is stabilized for storage in 650,000 bbl, cone-roofed tanks. Gases are transported to the power plant to fuel gas turbine generators. LPG is fractionated, chilled and stored in tanks adjacent to the crude oil tank farm from where it can be transported elsewhere or used as make-up fuel.

Diesel fuel is topped in the process area. Some is stored in tanks for local use, and the rest is transported by a six inch pipeline to the resupply base at Herendeen Bay. Primarily, the diesel is intended for the supply boats at Herendeen Bay and the tanker handling tugs at Lefthand Bay. Trucks, support vehicles and cargo handling equipment will also be diesel fueled.

Aviation fuel will be manufactured at the process area topping plant and transported by a four-inch pipeline to storage at Herendeen Bay. In the initial phase of the development, this fuel can be transported by truck to the airstrip. Storage tanks and a pipeline extension to the airstrip can be added later when air traffic increases.

Two tanker berthing trestles with mooring and breasting dolphins are shown for the 300 MBPD case. In this scheme, LPG would be loaded at one of the crude berths. Alternatively, LPG loading could be provided at the cargo dock if occupancy demand is too great at the crude berths. Even though there will be provision for fueling the tugs at the cargo dock, it would be preferable not to handle highly volatile hydrocarbons there.

The generalized flow diagrams for the onshore facilities are shown in Figure 8-17 . The crude storage tanks and loading lines are heat traced. In development of one of those sites where a trestle structure for tanker loading is not recommended, the loading line will be on the seabed and will not be heat traced. Neither would it be necessary to insulate the submarine line, because the pour point temperature of the crude is assumed to be less than the seawater temperature.

As indicated in the flow diagram, pipelined crude received from the offshore production platform is metered upon receipt at the process area. Stabilized crude and LPG are metered during delivery to the loading tankers.

The diesel and aviation fuel pipeline-transportation systems are not included in the flow diagrams, but these too are metered, both at delivery to the pipelines originating in the process area and at the receiving station in Herendeen Bay.

Tanker ballast water treatment is provided in order not to restrict the loading facilities to dedicated tankers. However, no provision is made for tanker bunkering. If, in operations planning for this terminal, it is considered desirable to add that capability, it can be done for the extra cost of fractionating bottoms and of providing storage and loading facilities at the berths.

The cargo dock is conceptually designed as a combination causeway and piled structure. Note that for the Lefthand Bay installation, some dredging at the face of the dock is deemed necessary to stabilize a steep seabed slope.

Tanker-handling tugs and oil-boom launches will customarily berth alongside the shoreward face of the dock. It was observed in Section 8.3 that severe southeast wind conditions might trouble small craft berthed at this location. Should this be so, one of the following options may be exercised:

- Construct fleet moorings at the head of the bay for temporary occupancy by small craft during severe conditions.

- Construct a permanent quay on the west side of the causeway for smallcraft berthing. This is not the primary recommendation, because winter northerlies channeled through the pass from Herendeen Bay may be more frequent and severe than southeast wave conditions.

Navigational aids will be required consisting principally of a headland light, buoys and range lights. In clear weather, tankers should be able to make port with no assistance other than docking assistance. However, the Alaska Peninsula is notorious for fog. Both pilotage and tug assistance from outside the bay may be prudent under such conditions; for at worst, only a swell would be running.

All ships approaching the port will be equipped with radar. But, considering the frequency of poor visibility conditions in this area, it may be desirable to establish a permanent, line-of-sight, horizontal positioning system for offshore navigation. This would assist ships in navigating through Unga and Korovin straits - especially the latter which is only 2 miles wide.

The number and sizes of the major terminal components are given in the following table.

	Number of Units	
	100 MBPD Throughput	300 MBPD Throughput
Storage		
650,000 bbl crude storage tanks	2	6
20,000 bbl LPG storage tanks	1	3
10,000 bbl diesel storage tanks	2	3
500,000 bbl ballast water storage tanks	1	2
Potable water tank	1	1
Tanker Loading		
Trestle and tanker berth	1	2
Loading arms	6	12
Royalty meter prover loop	1	1
Process Area		
Stabilization trains, 60 MBPD each with dehydrators	2	6
Fractionation trains, 100 MBPD each	1	3
LPG chill-down trains	1	3
Power Plant		
24 MW gas turbine generators	3	4

8.6 Resupply Base

The layout of the resupply base at the head of Herendeen Bay is shown in Figure 8-18. As can be seen in Figure 8-7, this resupply base is connected by road to Lefthand Bay and to the proposed airstrip on the east side of the bay. Since the base is part of a development complex which includes the terminal, it can share in the services provided by the terminal such as delivery of diesel and aviation gas by pipeline and electrical power through overhead lines.

Additional permanent living quarters are located at this base to house transients enroute by air to or from the offshore platform.

The water in Herendeen Bay is deep close to shore at the location selected for the base. This will permit supply boat berthing at a longshore quay constructed of concrete or earthfilled, sheet piling.

Travelling cranes on the quay will load the vessels with equipment and supplies delivered by truck from Lefthand Bay.

Although aviation fuel manufactured at Lefthand Bay will be delivered to storage tanks at the resupply base by pipeline, the fuel will be trucked to the airport on a demand basis. In future expansion of the facilities, a pipeline directly to storage tanks at the airstrip may be added.

Sea ice has not occurred in the Port Moller area in recent years, but it has been reported during severe winters in the past. The supply ships are assumed to have ice breaking capabilities. Nevertheless, it could become operationally burdensome if significant ice cover developed on Herendeen Bay and the adjoining area of the Bering Sea. Contingency plans should be developed for transferring resupply operations to Cold Bay or to Dutch Harbor during the one or two months of ice closure at Herendeen Bay.

The resupply base will not only be the primary port for all Bering Sea operations, but it will also serve an important role during offshore construction - especially of the offshore pipelines. Pipe will be delivered at this base and transported by pipehaulers to the laybarges.

8.7 Construction Planning

Construction planning for onshore facilities will be specialized to the area of Lefthand Bay and Herendeen Bay. The tank farm and tanker loading terminal together with the resupply base at Herendeen Bay can be treated as a single onshore complex.

The development of this complex must begin on a grassroots basis since there are no existing facilities. All principal construction equipment, materials and supplies will be brought in by ship.

Initially, this material will be lightered onshore at Lefthand Bay for lack of docking facilities. Emphasis will at first be placed on constructing the cargo dock at Lefthand Bay, the quay area at Herendeen Bay and the airstrip. Warehouses, docks, and quarters should be completed at the resupply base by the early part of the third year after start of the development in order to support offshore construction (see Schedule, Section 10). An earlier readiness may be necessary if the pipeline pulls through Herendeen Bay are carried out ahead of the first season of pipelay.

Ample time is available for completion of the main facilities at Lefthand Bay which are the crude storage tanks, process equipment, trestle structures and pipelines; since crude oil production does not begin until the sixth year at the earliest.

Further description of construction planning for onshore facilities is not necessary. Procedures are considered to be conventional and are taken into account in the development of the proposed construction schedule and cash flow diagrams of Section 10.3.

8.8 Engineering Considerations

8.8.1 Terminal and Supply Base Siting Alternatives

Site alternatives and components of an onshore tank farm and tanker loading terminal have been discussed above. However, specifics of this facility and the resupply base were treated in terms of a single site complex. The same basis is followed for construction planning of these facilities discussed in the previous section. If a different site is developed for the terminal, then the value of Herendeen Bay as a supply and construction base site is considerably reduced. For one, road connections between this port and another terminal would be long and through difficult terrain. For another, cargo unloading facilities would still have to be developed in Lefthand Bay in order to deliver ocean freight to the supply base. The one alternative of developing freighter docking facilities at Point Divide on the Hague Channel approach to Herendeen Bay (see Figure 8-18) has two drawbacks:

- Water depths in the Hague Channel off Port Moller are marginal with a large area sounding at 5 to 6 fathoms at mean lower low water. The cargo ships having access to the berth would be limited to about 15,000 deadweight tons with traffic only on the high tide. Otherwise, considerable channel development and maintenance dredging would be needed.

- Shipping from the Lower 48 would have to be detoured westward through the Unimak Pass and then eastward to reach the longitude of Port Moller.

If Herendeen Bay is not, at least, linked to sea access at Lefthand Bay, the supply base would be better situated at one of the sites described on Unalaska Island; unless, of course, construction and offshore supply takes place in the North Aleutian Basin. Then there would be no satisfactory alternative to Herendeen Bay as the site for a supply base, because supply boat travel distances would become too great.

As independent facilities, the supply base and terminal would accrue increased capital and operating costs, for even fuel would have to be brought in by sea to the supply base.

8.8.2 Terminal Cost Sensitivity to Siting

Construction costs at a terminal location different from Lefthand Bay may not vary significantly. The sites have different advantages and disadvantages as discussed earlier; and, to a large extent, these may offset each other. The on-land grading needed for the storage tanks of Lefthand Bay is not necessary at Cold Bay, Wide Bay or Fishrack Bay. But at the first two of these locations channel dredging is required, and at the third, a tunnel must be bored.

Similar comparisons and cost trade-offs can be recognized among the other sites.

8.8.3 Confirmation of Site Suitability

Preliminary site surveys are required at likely supply base and terminal locations to confirm their suitability in terms of ship access, nearshore water depths, and onshore foundation conditions. Once the preferred sites are tentatively confirmed, the application process for land acquisition, rights-of-way and permits should begin together with detailed surveys of the sites both onshore and offshore. This work would take place during the conceptual engineering phase of the development, and the results should be available for use in the detailed engineering phase of civil and structural design.

8.9 Summary and Conclusions

In review of possible onshore facility siting to service offshore platforms in the Bering Sea, it has been concluded that:

- The favored sites for a transshipment terminal are all situated on Unalaska Island. None of the three sites identified there is clearly preferred over the others for the reason that all lack sufficient land onshore for facility expansion. No terminal layouts or costs were prepared for this alternative.

- In the alternative of pipeline transportation of crude oil to an onshore storage and tanker loading terminal, Lefthand Bay of Balboa Bay on the Alaska Peninsula is favored over all other candidate sites.

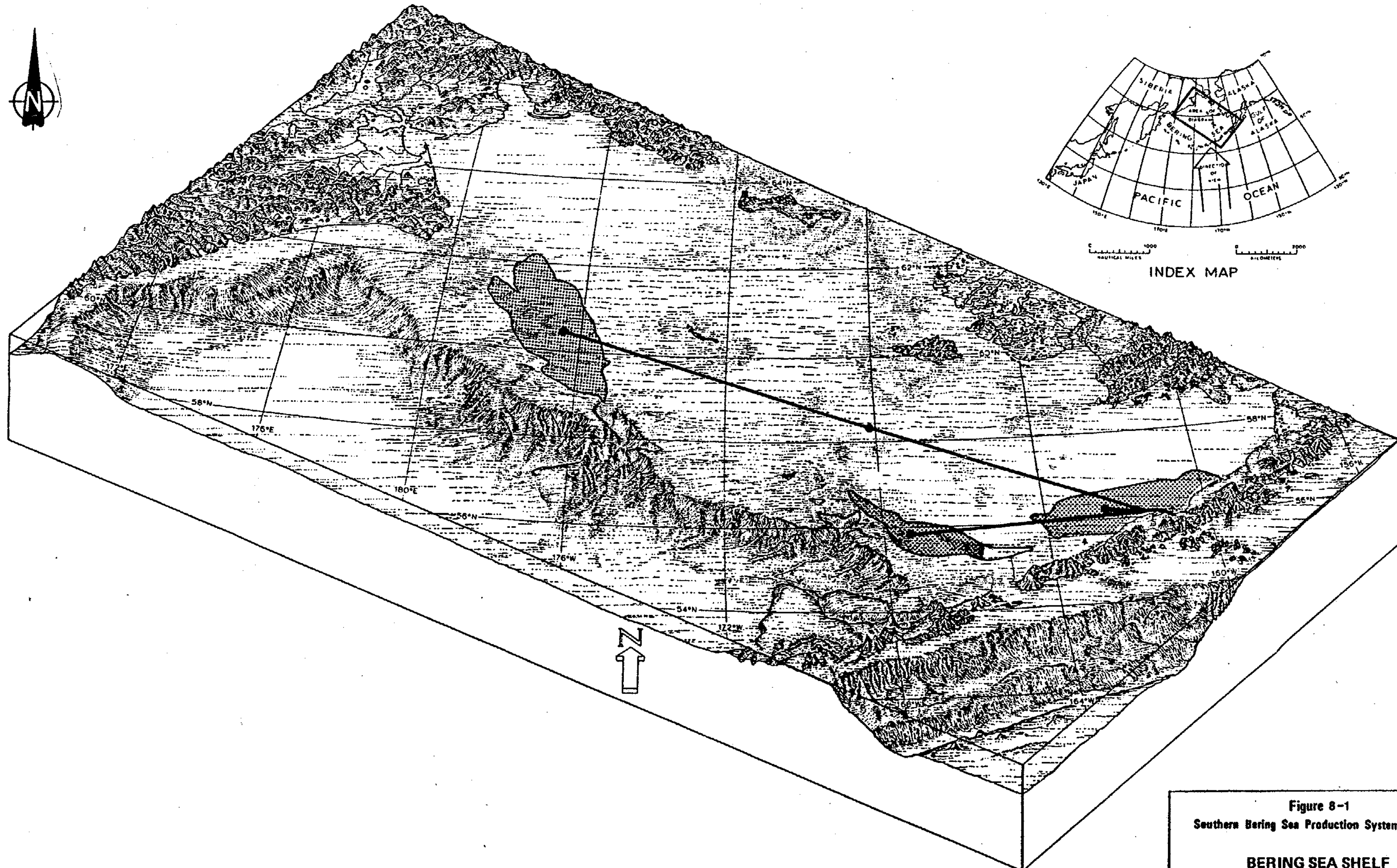
- The preferred siting of a construction and resupply base is contingent on the location of the tanker terminal.

  - If the terminal is on Unalaska Island, the supply base should be there too, but not necessarily on the same site as the terminal.

  - If the terminal is at Lefthand Bay, the supply base is recommended to be at Herendeen Bay.

- If the terminal is elsewhere on or near the Alaska Peninsula, a supply base to service any of the three lease areas can still be located at Herendeen Bay provided ship cargo handling facilities are located in Lefthand Bay. Otherwise, the supply base should be on Unalaska Island to service St. George or Navarin Basin platforms.

- To supply a platform in the North Aleutian Basin, the base would be at Herendeen Bay, even if the cargo dock had to be constructed in the Hague Channel off Point Divide.



ORTHOGRAPHIC DRAWING—
U. S. GEOLOGICAL SURVEY
OPEN FILE MAP 77–591

Figure 8–1
Southern Bering Sea Production Systems Study

BERING SEA SHELF
AND SURROUNDINGS

Job 15154 — BECHTEL — June 1982

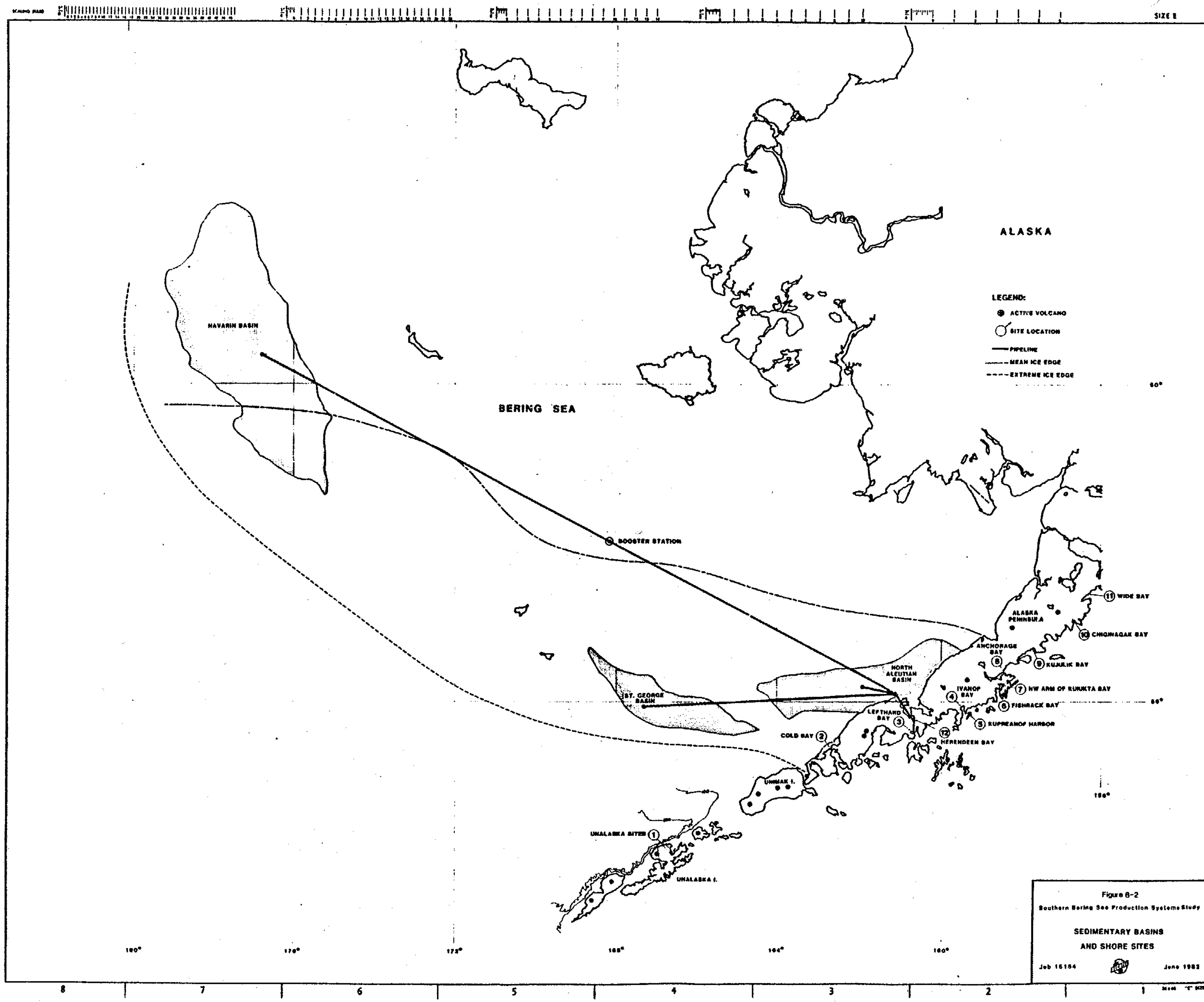


Figure 8-2

Southern Bering Sea Production Systems Study

SEDIMENTARY BASINS AND SHORE SITES

Job 15154 June 1982

FIGURE 8-3

COMPARISON AMONG CANDIDATE SITES FOR
TANK FARMS, TERMINALS AND RESUPPLY BASES

ITEM	Protection of Berth from Sea Waves	Space for Land Facilities and Future Expansion	Ship Berthing Potential		Submarine Pipeline Access	Land Pipeline Access	Tanker Access to the Berth		Protection from Wind at Berth	Proximity to Lease Areas	Suitability as a Tanker Transhipment Point	Suitability as an Offshore Construction or Platform Resupply Base	Proximity to an Offshore Resupply Base
			Deep water near shore	Turning Basin			Harbor Approaches	Offshore					
(1) UNALASKA ISLAND AREA	Excellent	Good	Excellent	Excellent	Poor	N.A.	Excellent	Excellent	Good	Excellent	Excellent	Excellent	Excellent
(2) COLD BAY	Excellent	Excellent	Good	Excellent	Excellent	Excellent	Poor	Poor	Poor	Good to Excellent	Good	Good	Good
(3) LEFT HAND BAY	Excellent	Excellent	Excellent	Excellent	Excellent	Excellent	Excellent	Excellent	Good	Good	Good	Poor	Excellent
(4) IVANOF BAY	Excellent	Excellent	Excellent	Excellent	Good	Poor	Excellent	Excellent	Good	Good	Good	Poor	Poor
(5) KUPREANOF HARBOR	Good	Good	Excellent		Poor	N.A.	Good	Excellent	Poor	Good	Poor	Poor	Poor
(6) FISHRACK BAY	Excellent	Poor	Good	Good	Good	Poor	Excellent	Excellent	Good	Good	Poor	Poor	Poor
(7) NW ARM OF KUIUKTA BAY (PORTAGE BAY)	Excellent	Good	Excellent	Good	Good	Poor	Good	Excellent	Poor to Good	Good	Poor	Poor	Poor
(8) ANCHORAGE BAY	Excellent	Poor	Excellent	Poor	Good	Poor	Good	Excellent	Good	Good	Poor	Poor	Poor
(9) KUJULIK BAY	Poor	Excellent	Good	Excellent	Good	Good	Excellent	Excellent	Poor	Poor	Poor	Poor	Poor
(10) CHIGINAGAK BAY	Poor	Excellent	Good	Good	Good	Poor	Good	Good	Poor	Poor	Poor	Poor	Poor
(11) WIDE BAY	Excellent	Excellent	Good	Excellent	Good	Poor	Good	Excellent	Poor	Poor	Poor	Poor	Poor
(12) HERENDEEN BAY	Excellent	Excellent	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	Good to Excellent	Poor	Excellent	N.A.

FIGURE 8-4
SITE SELECTION

ITEM	Tank Farm/Tanker Terminal Site with a Submarine Pipeline to Shore	Tank Farm/Tanker Transhipment Terminal— No Pipeline to Shore	Supply Boat Base for offshore Construction and Platform Resupply
① UNALASKA ISLAND	Poor	Excellent	Excellent
② COLD BAY	Good	Good	Good
③ LEFTHAND BAY	Excellent	Good	Poor
④ IVANOF BAY	Good	Poor	Poor
⑤ KUPREANOF HARBOR	Poor	Poor	Poor
⑥ FISHRACK BAY	Poor	Poor	Poor
⑦ NW ARM OF KUIUKTA BAY (PORTAGE BAY)	Poor	Poor	Poor
⑧ ANCHORAGE BAY	Poor	Poor	Poor
⑨ KUJULIK BAY	Poor	Poor	Poor
⑩ CHIGINAGAK BAY	Poor	Poor	Poor
⑪ WIDE BAY	Poor	Poor	Poor
⑫ HERENDEEN BAY	Poor	Poor	Excellent

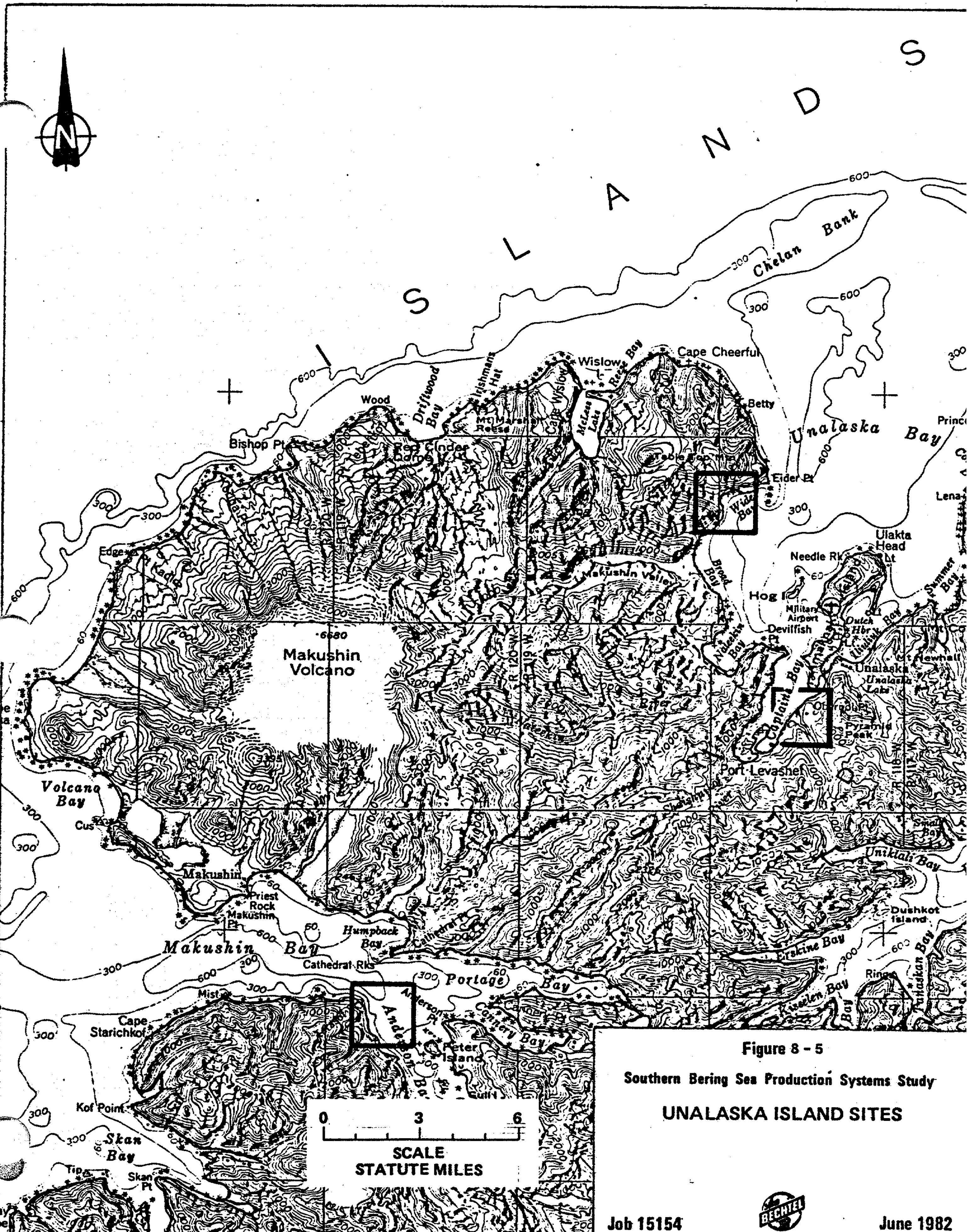


Figure 8 - 5

Southern Bering Sea Production Systems Study

UNALASKA ISLAND SITES

Job 15154 **June 1982**

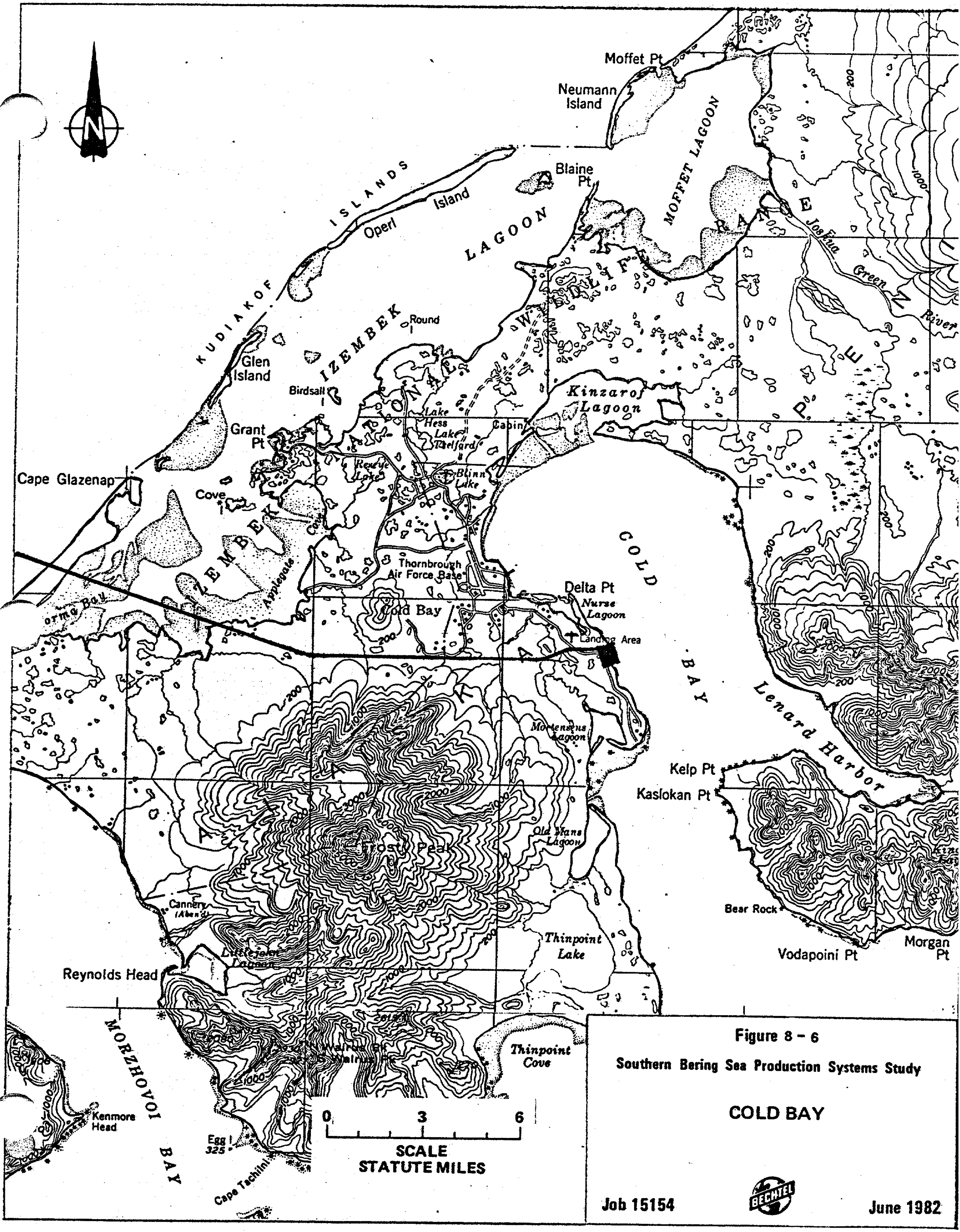


Figure 8 - 6

Southern Bering Sea Production Systems Study

COLD BAY

Job 15154 BECHTEL **June 1982**

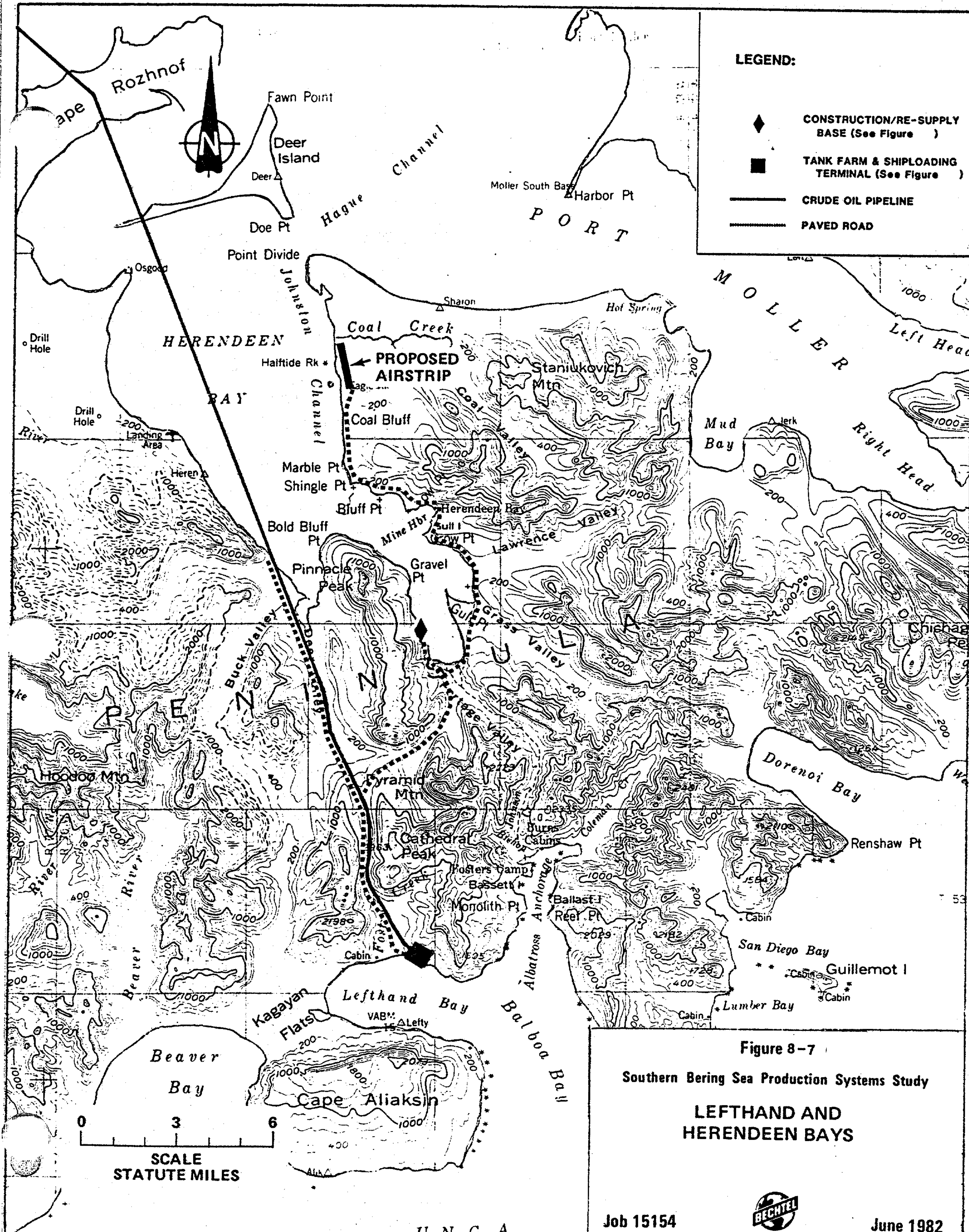


Figure 8-7

Southern Bering Sea Production Systems Study

LEFTHAND AND HERENDEEN BAYS

Job 15154

June 1982

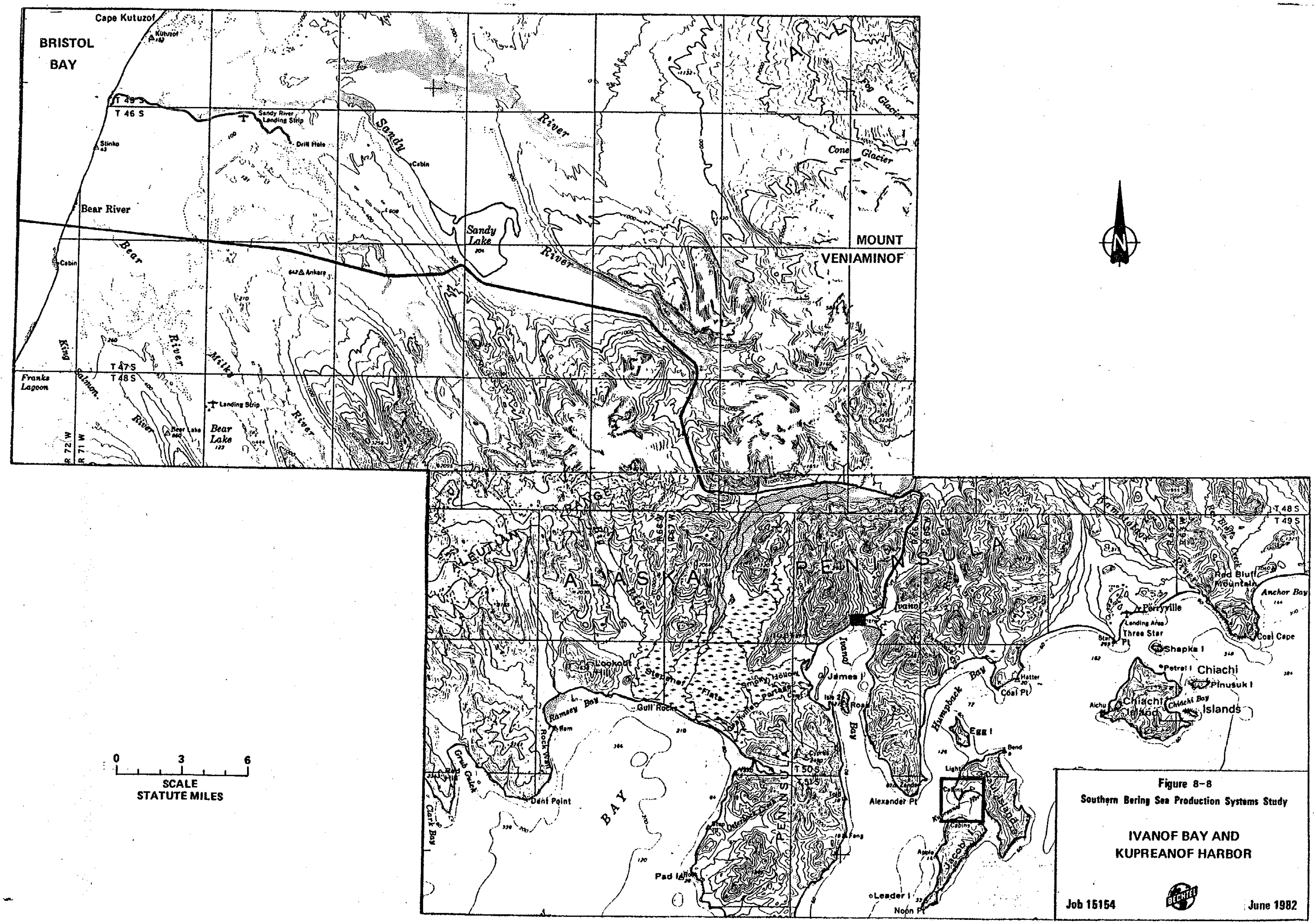


Figure 8-8
Southern Bering Sea Production Systems Study

IVANOF BAY AND
KUPREANOF HARBOR

Job 15154

June 1982

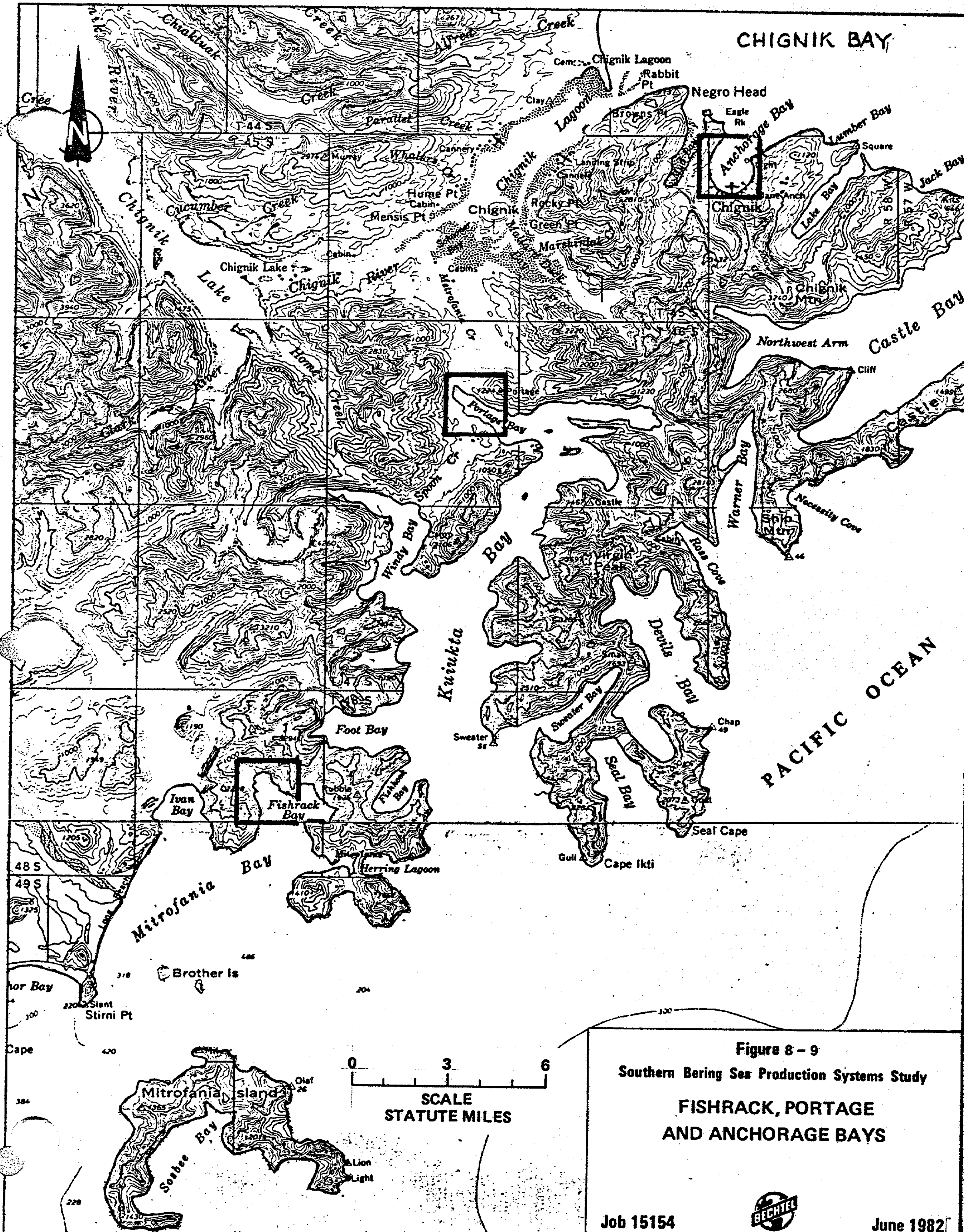


Figure 8 – 9

Southern Bering Sea Production Systems Study

FISHRACK, PORTAGE AND ANCHORAGE BAYS

Job 15154 BECHTEL **June 1982**

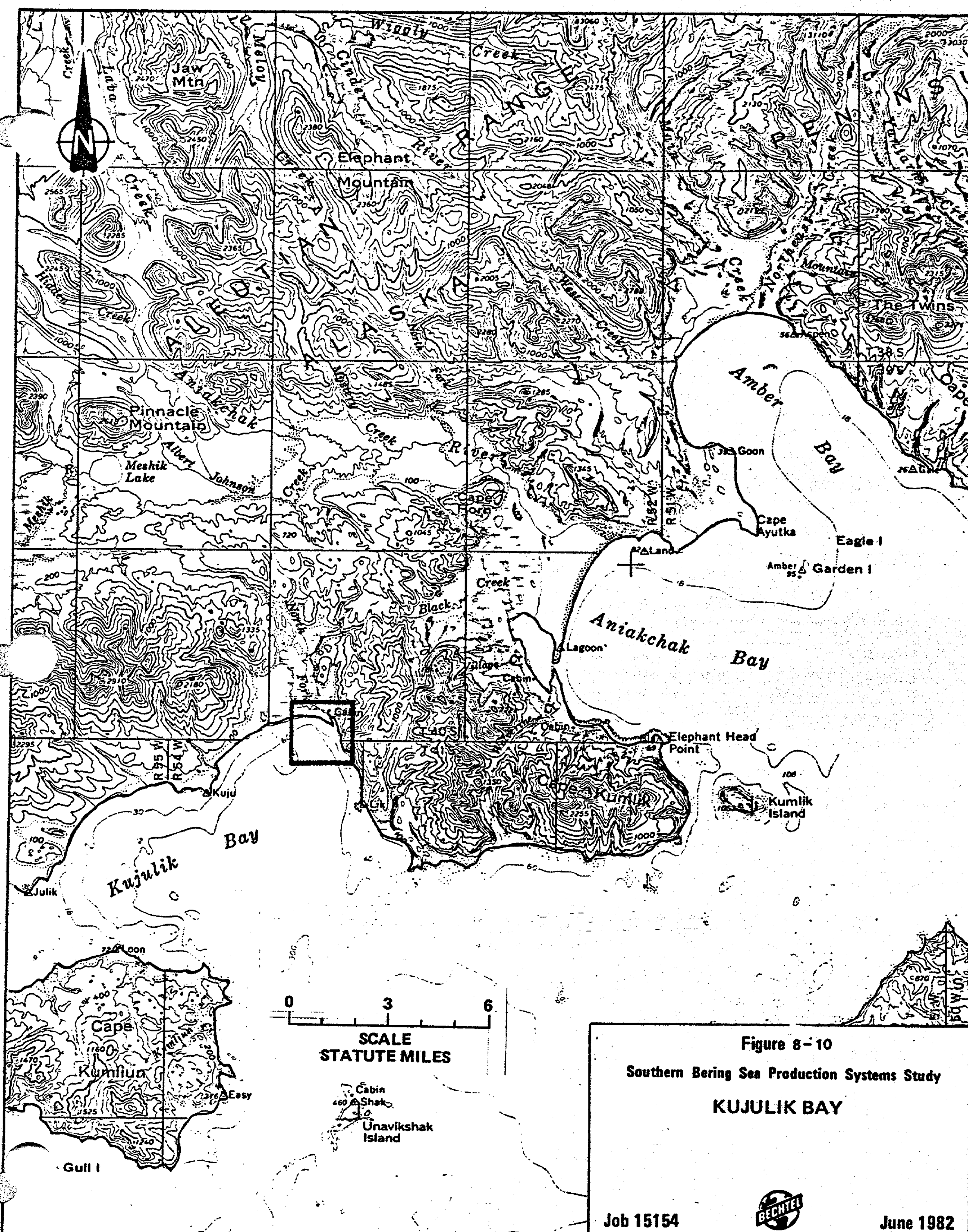


Figure 8-10

Southern Bering Sea Production Systems Study

KUJULIK BAY

Job 15154

June 1982

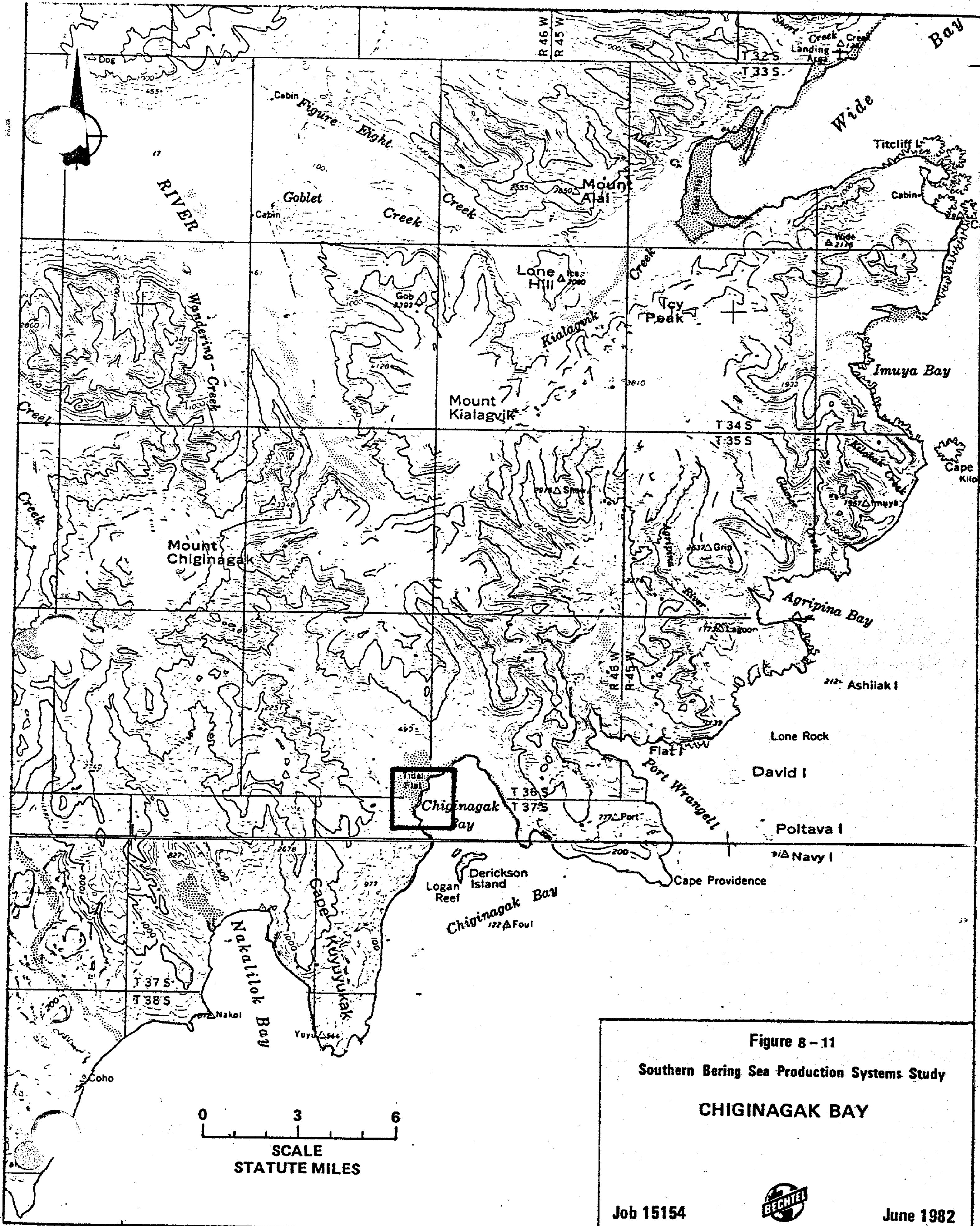


Figure 8 – 11

Southern Bering Sea Production Systems Study

CHIGINAGAK BAY

Job 15154 BECHTEL **June 1982**

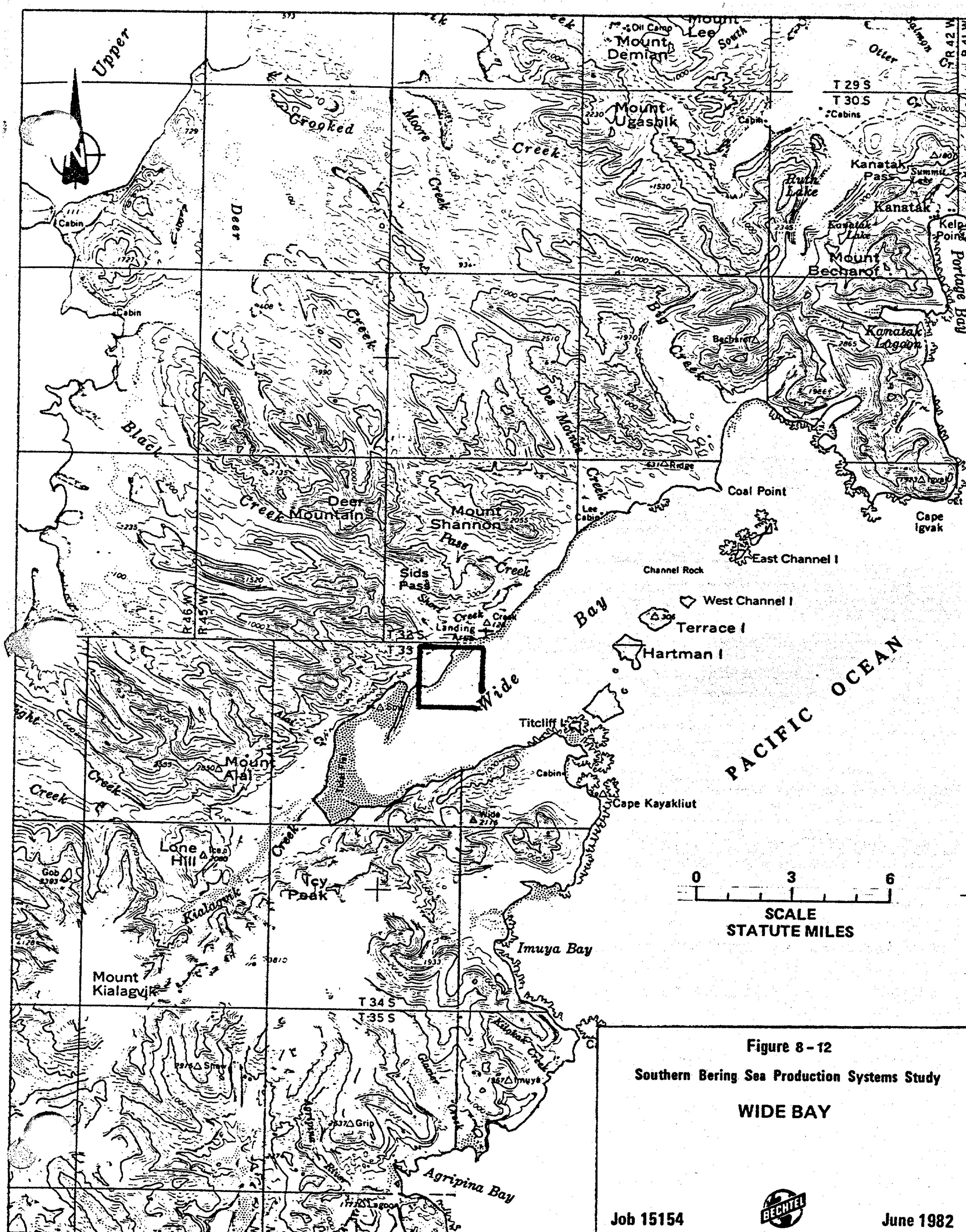


Figure 8-12

Southern Bering Sea Production Systems Study

WIDE BAY

Job 15154

BECHTEL

June 1982

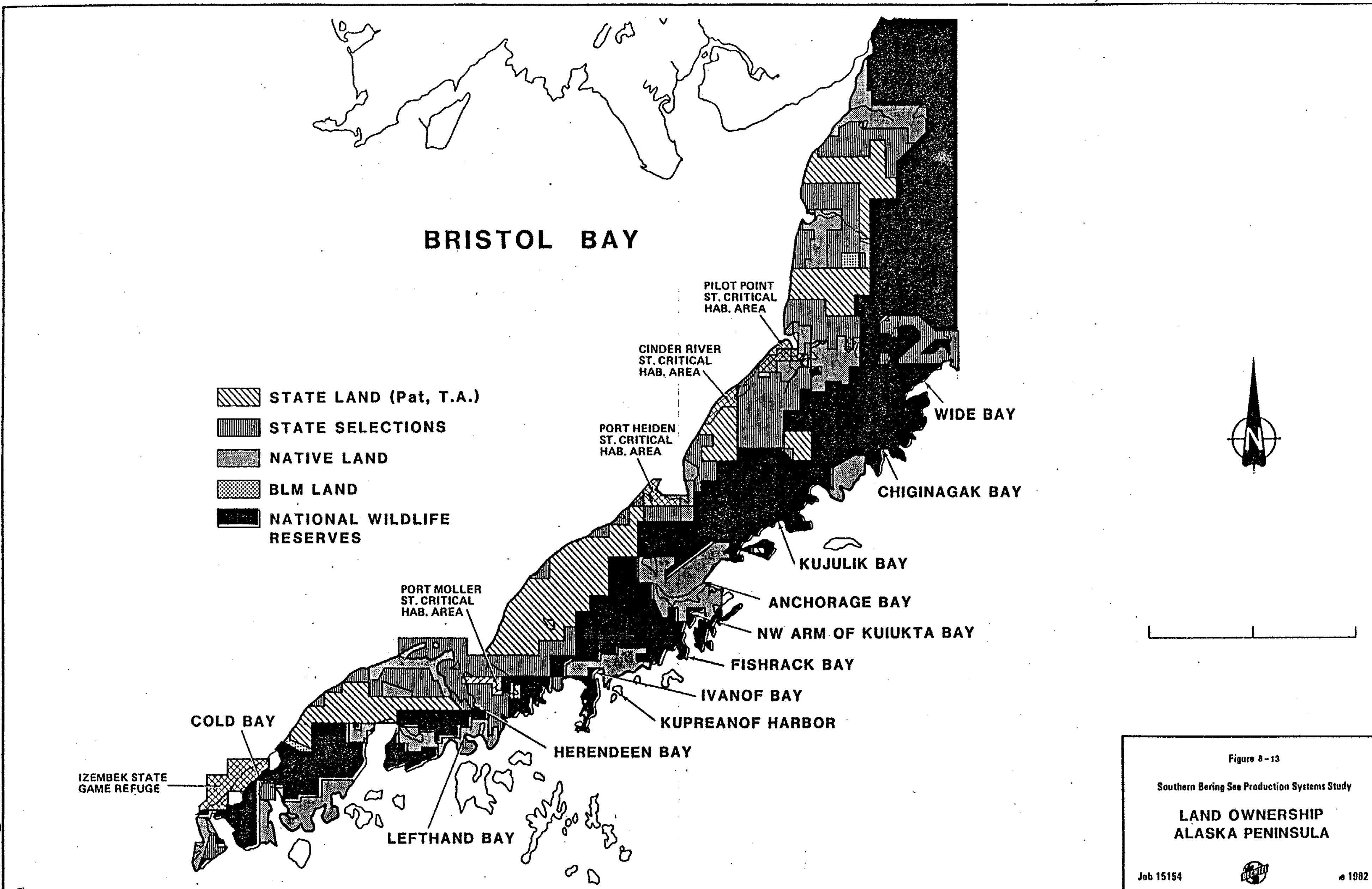


Figure 8-13

Southern Bering Sea Production Systems Study

LAND OWNERSHIP
ALASKA PENINSULA

Job 15154 © 1982

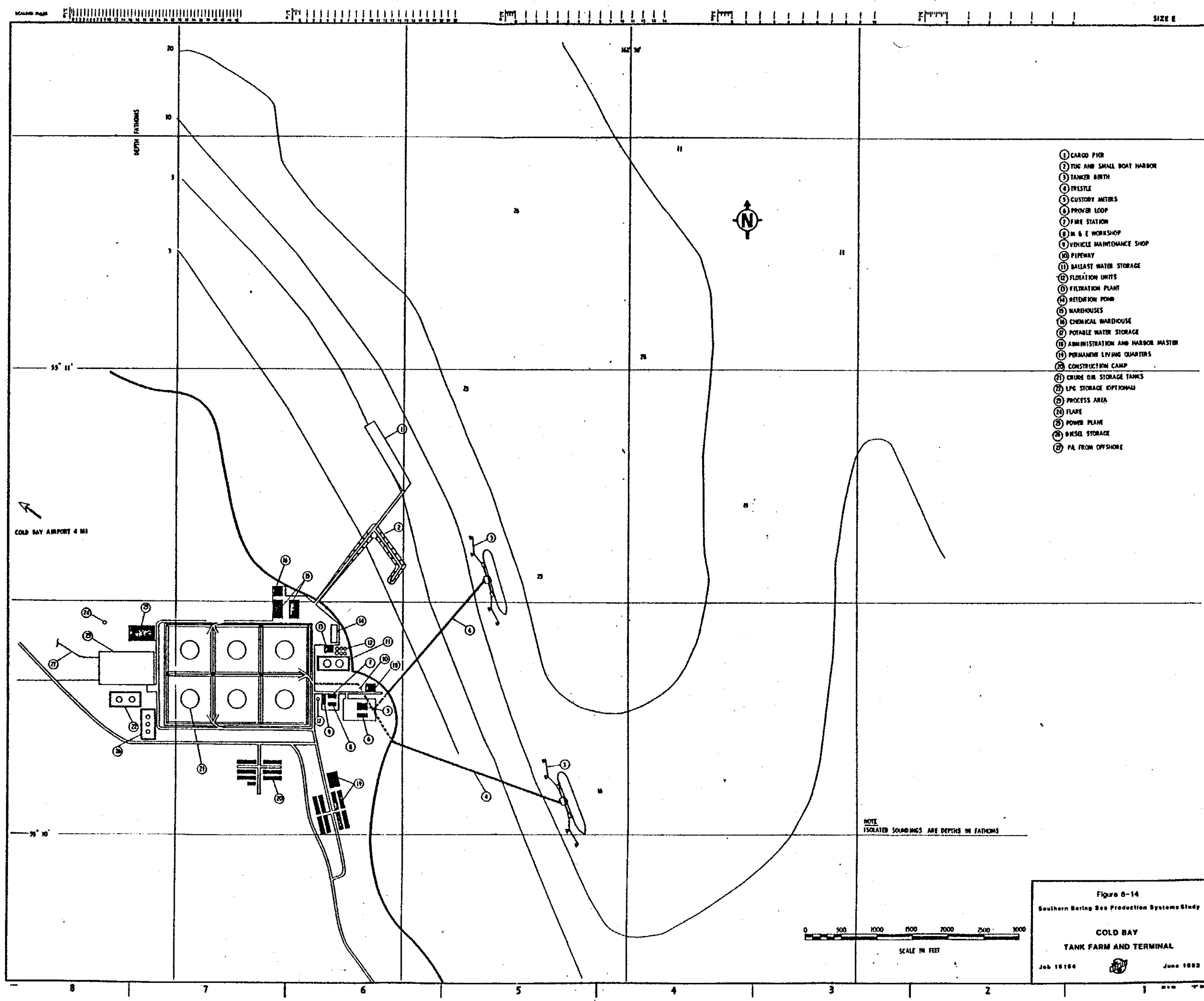


Figure 8-14

Southern Bering Sea Production Systems Study

COLD BAY

TANK FARM AND TERMINAL

Job 15154 June 1982

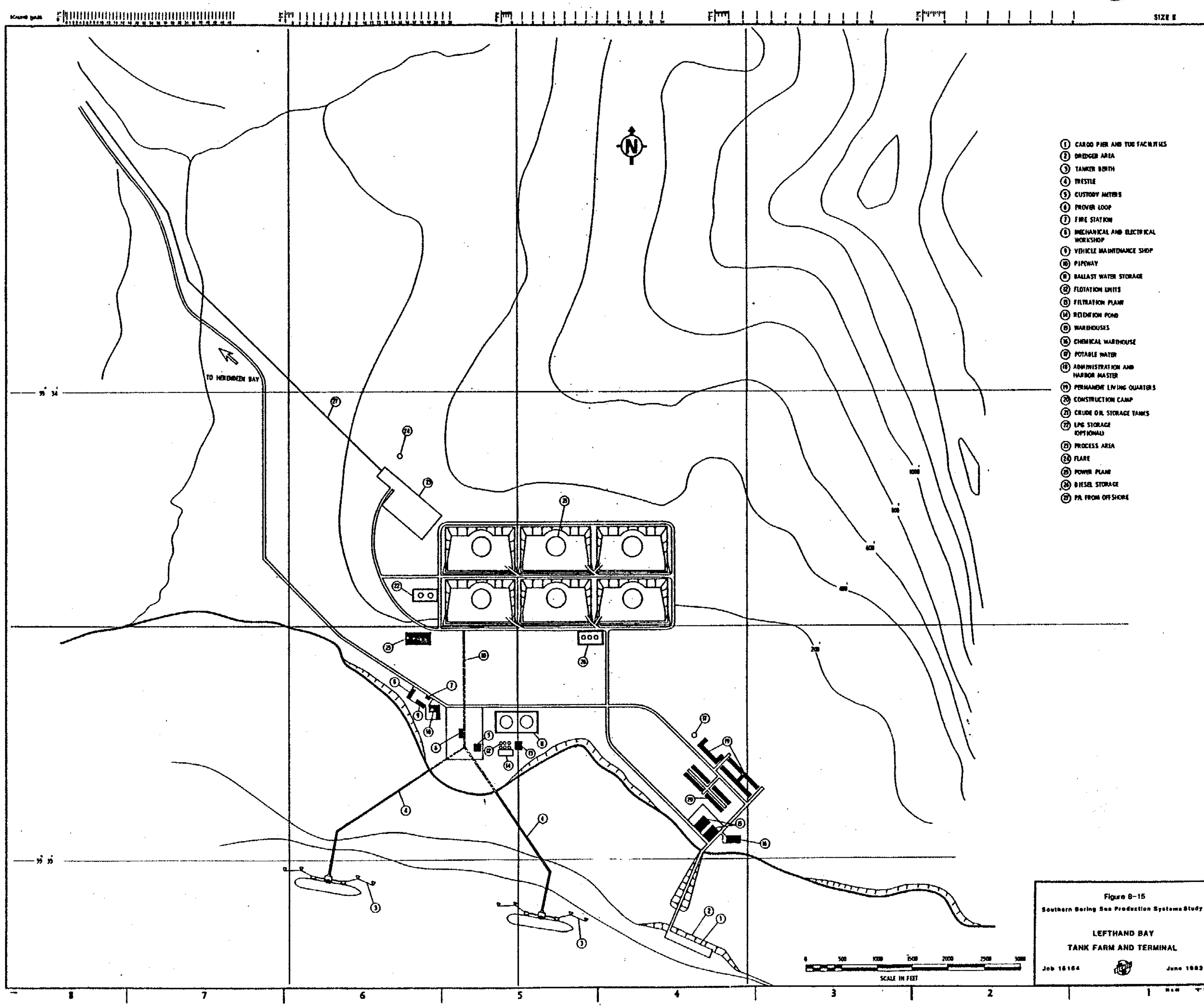


Figure 8-15
Southern Bering Sea Production Systems Study
LEFTHAND BAY
TANK FARM AND TERMINAL
Job 16164 June 1982

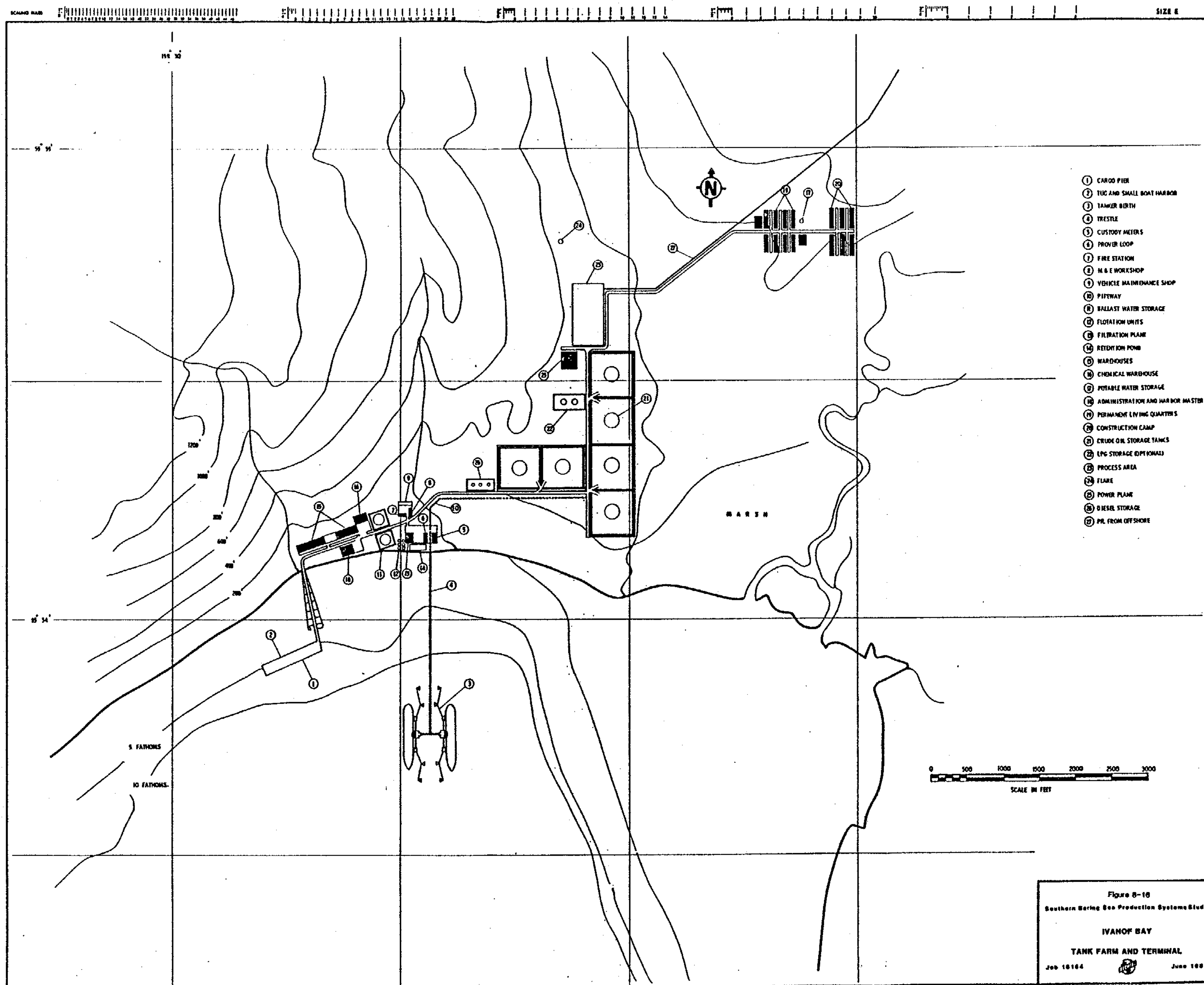


Figure 8-16

Southern Bering Sea Production Systems Study

IVANOF BAY

TANK FARM AND TERMINAL

Job 16164 June 1982

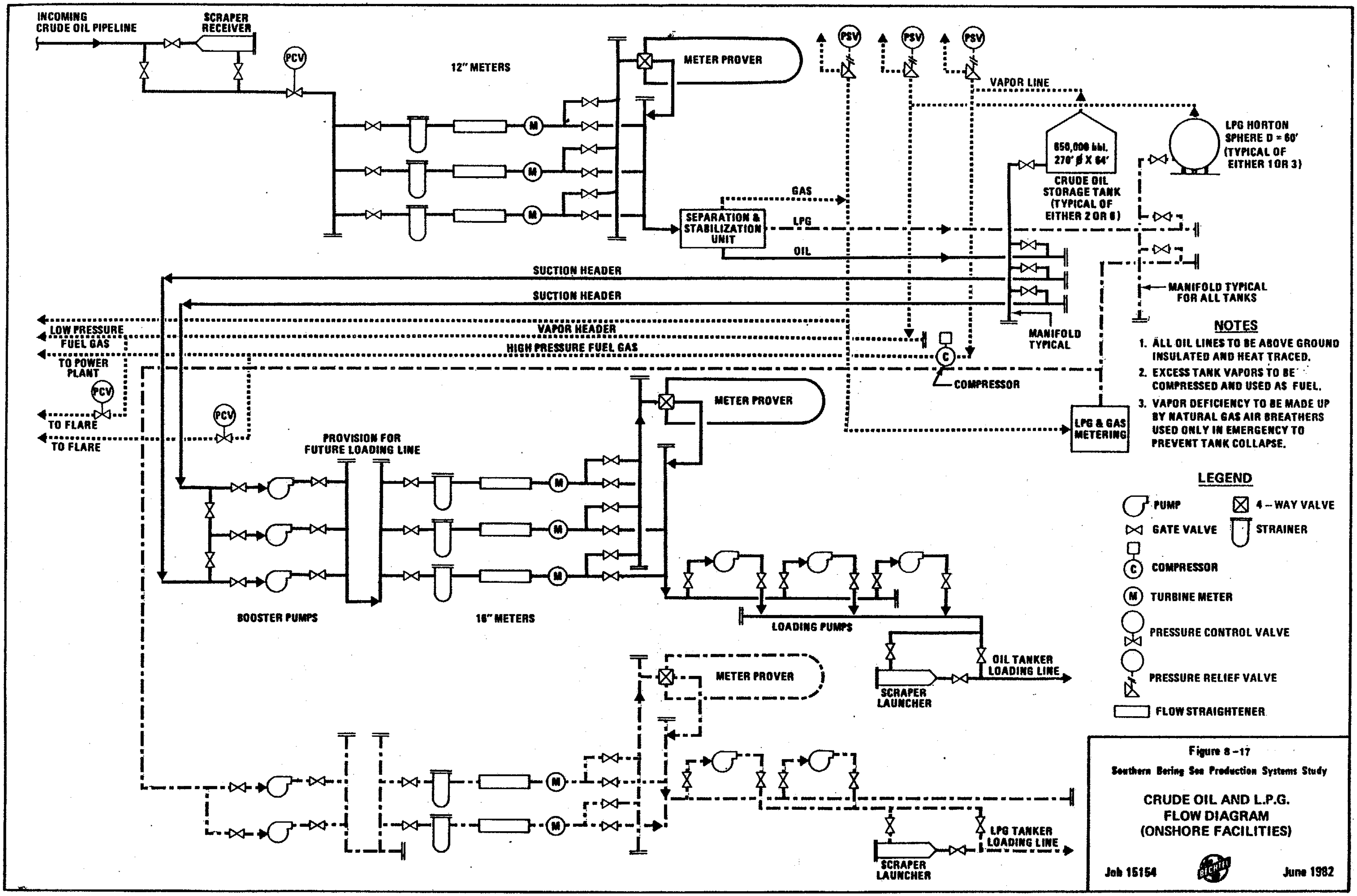


Figure 8-17

Southern Bering Sea Production Systems Study

CRUDE OIL AND L.P.G. FLOW DIAGRAM (ONSHORE FACILITIES)

Job 15154 June 1982

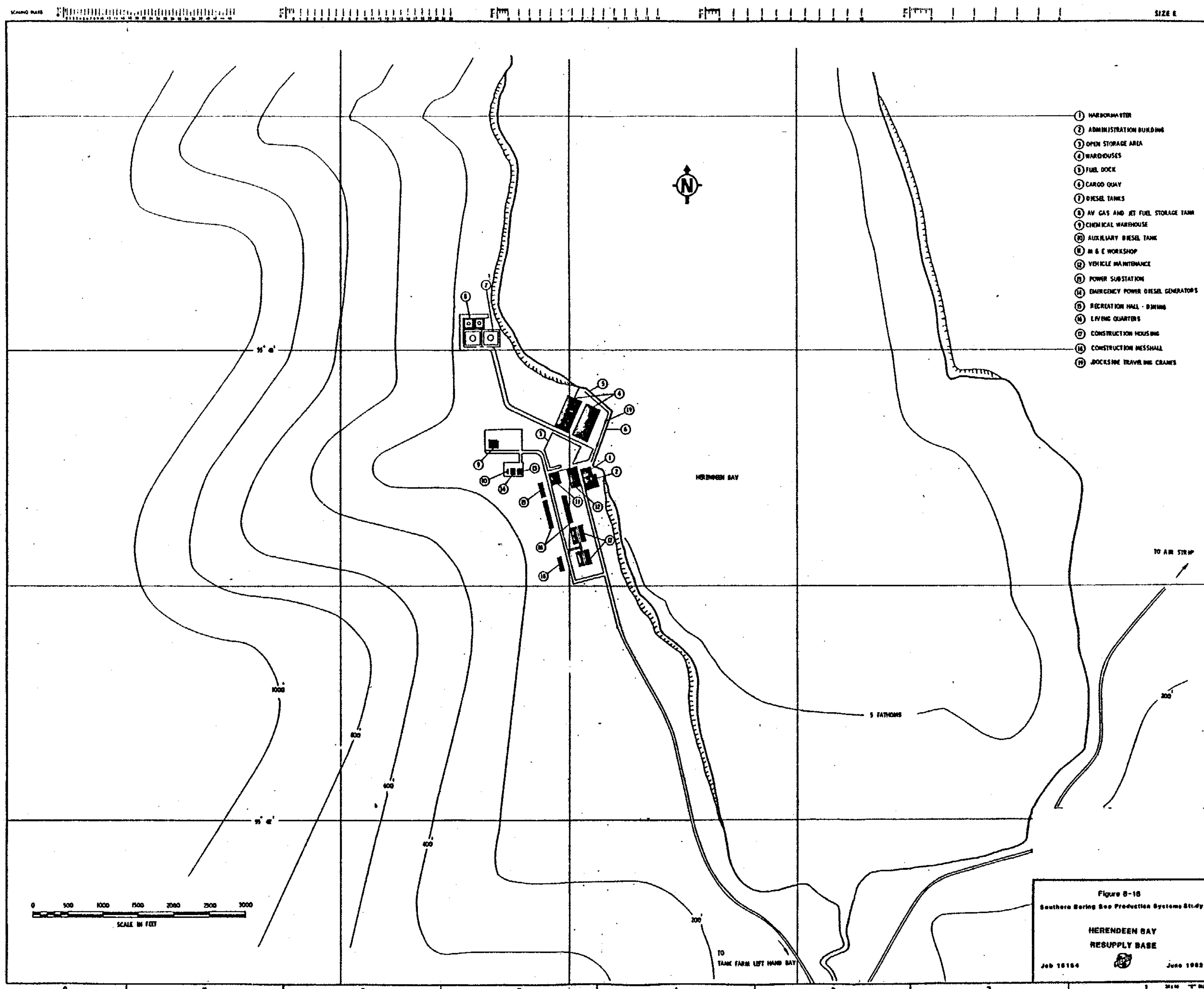


Figure 8-16
Southern Bering Sea Production Systems Study
HERENDEEN BAY
RESUPPLY BASE
Job 16154 June 1982

TABLE OF CONTENTS

SECTION 9.0

LIST OF FIGURES

SECTION 9.0

LIST OF TABLES

SECTION 9.0

9.0 OFFSHORE STORAGE AND LOADING STRUCTURE

9.1 Scope of Work

As an alternative to the transportation of crude oil via submarine pipeline to a shore terminal on the Alaska Peninsula, an offshore storage and tanker loading terminal was briefly investigated. The scope of work for this portion of the study was the following:

- Evaluate possible offshore storage and loading structure alternatives.
- Define primary topside mechanical requirements.
- Develop a preliminary design for each of the three lease sale area base case conditions for a range of two storage volumes.
- Identify any engineering problems or constraints.
- Develop cost estimates and schedules for purposes of lease sale planning.

9.2 Design Criteria and Assumptions

- Crude storage should be provided for 7 to 10 days of production at rates from 100,000 BPD to 300,000 BPD.

- The terminal should be capable of year-round operations even in periods of ice coverage.

- Ice breaker operations are available on a year-round basis to assist marine operations in periods of heavy ice coverage.

- A dedicated fleet of ice strengthened tankers in the range of 100,000 to 150,00 DWT is utilized.

9.3 Structural Alternatives Considered

9.3.1 General

Several alternative offshore storage and/or loading structures were considered and evaluated for each of the three basin sites and two storage capacity requirements.

9.3.2 Cellular Concrete Gravity System

Underwater storage structures able to accommodate up to 3,000,000 bbl of crude oil storage have been studied. Both steel and concrete structures were considered. For large storage capacity, prestressed concrete structures appear to provide greater economy than steel structures. Therefore, the study concentrated on concrete storage structures.

The first structure considered was the North Sea type concrete storage structure (Figure 9-1). This structure consists of a series of interconnected vertically positioned hexagonal tanks surrounding a central shaft that extends to the surface to support the transfer pumps, instrumentation and ballast control system.

Transfer of the crude oil to the tankers can be accomplished through a separate SPM as discussed in Section 9.3.5 or through an integrated SPM as shown on the figure. Storage capacity can range from one to three million barrels. Insufficient floating stability may present transportation and installation difficulties. This problem can be resolved by means of heavy ballasting or auxiliary temporary buoyancy.

9.3.3 Tank Type

For shallow water, it may be practical to provide storage capacity in a large tank structure as shown in Figure 9-2. Large wave and ice forces can be developed and must be resisted by the gravity structure. Because of the sea ice, it may not be practical to employ a perforated breakwater wall at sea level to reduce wave forces. Loading from the tank structure would be accomplished through a separate SPM facility adjacent to the storage tank as discussed in Section 9.3.5.

9.3.4 Floating Storage Concepts

A floating concrete caisson as shown in Figure 9-3 was evaluated. A series of cylindrical tanks surrounding a central tower containing a conical shape near the water line to reduce ice loads acting on the tower would be employed. As in the underwater storage alternatives, an integral SPM could be used to load out crude oil to oil tankers.

To position the floating structure, a multi-line mooring system is used to resist wind, wave and ice forces from all directions. A flexible riser hose is used to connected the submarine pipeline to the floating structure.

This alternative is particularly suitable to deep water for the mooring lines to effectively anchor the floating structure. Ice loads acting on structure may induce a sustained tilt to the structure with corresponding large mooring forces.

3.5 Single Point Tanker Loading Structure

Single point moring concepts such as the SALM, CALM, or some other hybrid structure are potential product offloading system alternatives for use in the Southern Bering Sea. These offloading structures are field proven in such harsh environments as the North Sea. The existence of sea ice in the Bering Sea, however, poses new problems for SPM's.

A number of potential SBM designs for a sea ice environment are included in Section 3.5.5.3. As these designs were proposed for use in conjunction with the floating production systems evaluated in this study, the designs are based upon seasonal terminal operations. These SPM's would be designed for local ice loads; however, the actual ice resistance would be compatible with that of the floating production platform. If these design ice conditions are exceeded, the floating platform would abandon the site and the SPM would either the submerged or disconnected and towed to safety. The main point here is that the ice strength of the offloading system is compatible with that of the entire floating production system.

Significant design modifications would be required for year-round SPM terminal operations. The SPM must be able to withstand maximum expected local ice impact loads. In addition, for ice coverages of 5/8 or greater, the SPM must have the capability to remain operational after failing the sheet ice. A candidate design for such conditions is a modified SALM/CALM hybrid concept, as shown in Figure 9-4. The upper section of the single anchor leg would be ice-strengthened to withstand the maximum expected ice loads. In order to develop the required restoring force in the mooring system, guylines would be attached to the structure with fairleads below the ice level as shown and anchored to the seabed. The anchor system would either be drag anchors or possibly anchor piles if required. In order to resist possible maximum ice forces of about 2,000 to 3,000 k, a major guyline system utilizing approximately twelve bridge strand guylines of about 4-1/2 in. diameter would be required.

Two strategies could be employed. The first would utilize buoyancy as the normal supporting mechanism with fully preloaded guylines only necessary for maximum ice conditions. An alternate would be to have no significant buoyancy (which would possibly reduce the diameter and corresponding ice loads) and rely on guylines for all support. Clump weights could provide protection against tanker override.

During period of ice coverage, ice-breaking workboats could be used to break-up and deflect the ice around the mooring terminal.

The main advantage of this single offloading system is relatively low cost and minimal susceptibility to damage due to potential tanker override on the mooring. The main disadvantage to this system is that it is not field proven in an ice environment. A higher level of confidence in predicting maximum ice loads is required as this specific design is quite sensitive to ice loads.

9.3.6 Conclusions

The alternative selected for all basins is the Condeep type storage structure having an integrated rigid SPM. The main reasons in support of this selection are:

a. Minimum environmental loads: The base caisson is completely submerged and thus environmental loads due to interactions with ice and waves are minimized.

b. Proven construction method: The knowledge and experience necessary for the design and construction of this prototype subarctic structure are available. Based on the performance of this structure, subsequent designs can be varied.

A possible disadvantage associated with the selected alternative is the lack of redundancy during potential collisions with tankers. The probability of the occurrence of such collisions can be significantly reduced if stringent docking practices are utilized. It is also possible to design the main shaft to resist damage for tanker impacts at low velocities, and with a relatively small diameter single shaft the possibility of a direct collision is reduced.

9.4 Topside Requirements

Topsides for the offshore storage and loading structure having crude oil storage in its base are shown in plan in Figure 9-5. Only one level is required on a deck structure of about 100 ft. x 100 ft.

The principal components are power generators, oil-water separation equipment and living quarters for several operators.

Water treatment facilities are used for the ballast in the crude oil storage cells; the treated water with oil removed is discharged into the sea.

Power generation is indicated in the figure to consist of two turbo-generators, one on standby. Fuel is received by submarine gas line from the production platform. An alternative arrangement would be to provide the loading structure with electric power from the production platform, in which case diesel generators for emergency power and a diesel storage tank would occupy the space provided for the turbogenerators.

Because of the mechanical and electrical interfaces with the base, the topside is assumed to stay fixed. This requires that the tanker loading arm traverse around the topsides as discussed in Section 9.5.

9.5 Tanker Mooring and Cargo Transfer

It is assumed that the shuttle tankers are are in dedicated service and are specially outfitted for off-loading from a single point mooring. Bow controls and turbines capable of maintaining a constant backtension on the mooring hawser are required. It is further assumed that an icebreaker is in attendance to break up sea ice and thereby minimize ice loads on the moored shuttle tanker.

The alternative which was selected for evaluation and is the basis of the cost estimates employs a rigid (nonarticulated) single point mooring integral with the gravity base. The tanker is assumed to weathervane around the single point mooring in response to net environmental loads.

A revolving gantry style loading arm was the primary system considered. This employs a large trussed boom which revolves in a system of roller bearings at the top level of the deck. The tip of the boom extends well beyond the perimeter of the deck. The cargo transfer hoses are passed from the tip of the boom to the ship's bow. The revolving mechanical interface could employ articulated joints or flexible hose made up to a manifold. The mooring hawser would travel on a track mounted to the exterior of the main shaft below the deck level. Some system of shock absorbtion should be considered in the mooring hawser to minimize dynamic loads on the rigid SPM.

9.6 Preliminary Designs

A limited preliminary design of the selected concept was performed for each of the lease sale basins. All base case environmental criteria and water depths were used for purposes of the preliminary design and cost estimates. No actual analysis was performed for the evaluation of the storage structures. The discussion concerning the selected alternative is provided using a direct comparison between the storage structure and the similar drilling production structures. The following environmental loading conditions were assumed:

- Ice Loads - 25 percent of that predicted for the drilling and production structures. Note: During tanker moorings it is assumed an ice breaker is breaking up the sheet ice in the area. Even if some tanker mooring loads were added to the above value, this is not expected to be a controlling case.

- Wave Loads - It was conservatively assumed that the applied wave forces were the same as for the concrete drilling and production platforms with similar base dimensions. Note: This may be slightly conservative due to the reduced loads on the shaft.

- Seismic Loads - It was conservatively assumed that the applied seismic loads were the same as for the concrete drilling and production platforms with similar base dimensions. Note: This may overestimate base shear by 20 to 30 percent due to the reduced structural and topside weight.

6.1 Storage and Loading Structure for St. George or Navarin

The base of the proposed structure is hexagonal in plan, having gross dimensions of 570 feet by 680 feet. It is to be founded in 450 feet of water. The base caisson height can be varied depending upon the necessary storage capacity. A caisson height of approximately 200 feet will be required to provide sufficient storage space for 3 million barrels. See Figure 9-6. This height can be reduced to approximately 110 feet if the required storage capacity is only 1 million barrels. The structure's single shaft is tapered, having a diameter of 75 feet at the bottom and 40 feet at the mean sea level. The height above the mean sea level should be determined based on the tanker clearance and the maximum sea state associated with this requirement.

The global geotechnical stability can be conservatively described by the safety coefficients obtained for the drilling production structures given in Section 5.3.6.3. The lower storage capacity (1×10^6 BBL) corresponds to the base case in each basin and the resulting safety coefficients satisfy the design criteria. The larger storage capacity (3×10^6 BBL) corresponds to the case of storage conditions variation. For this case in the St. George Basin the seismic design criteria in bearing was exceeded by 25 percent. It is possible that a storage structure, unlike the drilling production structure, will satisfy the minimum bearing stability requirement. This is because of the assumed reduction in base shear and overturning due primarily to much lighter topsides. The design criteria for the larger drilling production structure in the Navarin Basin was fully satisfied and thus does not present any difficulties for the storage structure. Material takeoff is shown in Table 9-1.

9.6.2 Storage and Loading Structure for North Aleutian

The alternative selected for the North Aleutian Shelf has a configuration similar to those of the storage structures for the St. George and Navarin basins (see Figure 9-7). The base case is to be founded in 300 feet of water and the base plan dimensions are 440 feet by 530 feet. Also here, the caisson height can be varied to accommodate the required storage capacity. A caisson height of approximately 110 feet is sufficient to provide storage space for 1 million barrels. If the caisson height is increased to 200 feet, the structure is able to accommodate approximately 2 million barrels. It should be noted that the minimum caisson height (110 feet) was determined from structural integrity requirements. If it is mandatory to accommodate 3 million barrels, then the base plan dimensions should be increased. This option is preferred over the increase in caisson height. The structure's single shaft is a vertical circular cylinder having an outside diameter of 40 feet tapering to 70' at the base. As before, the height above the mean sea level should be determined by the tanker clearance and the maximum sea state associated with this requirement.

The global geotechnical stability can be conservatively assessed using the safety coefficients for the corresponding drilling production structures as shown in Section 5.5.6.3.

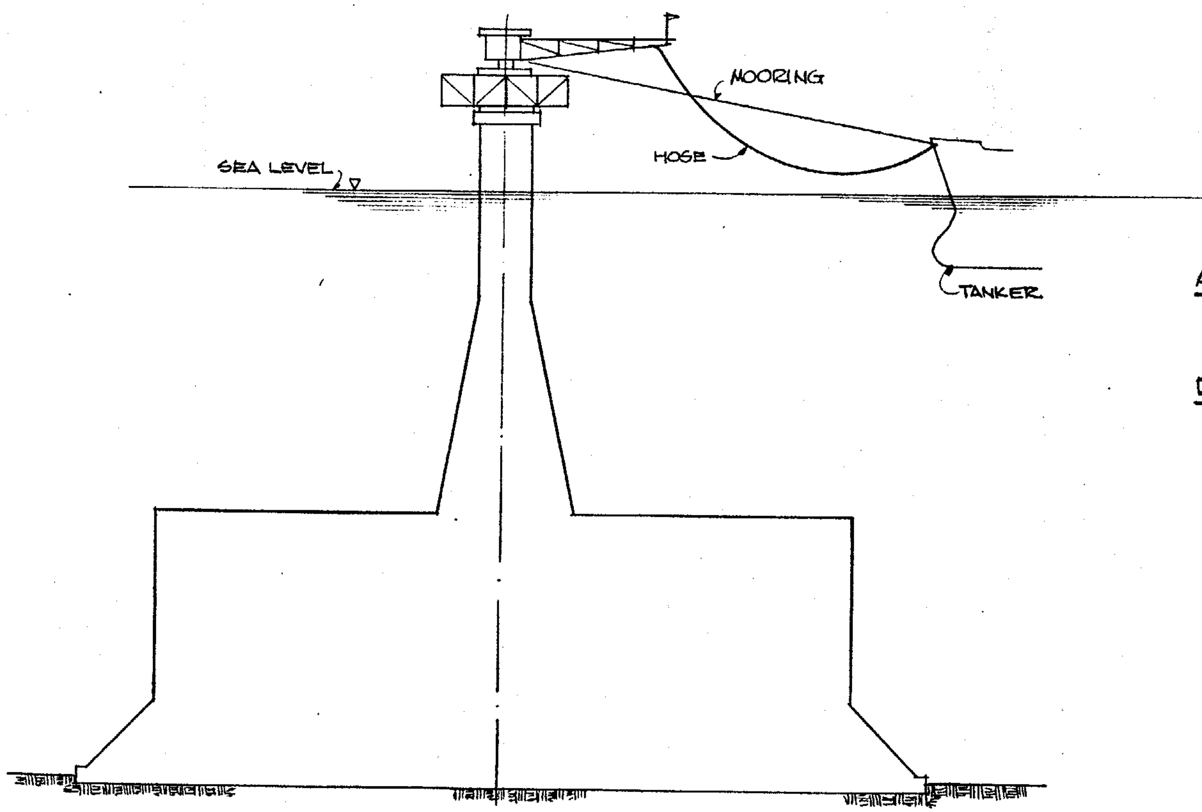
Material take-off is shown in Table 9-2.

From these coefficients it can be observed that the minimum stability criteria is fully satisfied for the base case foundations.

9.7 Conclusions

Based on a limited investigation of concrete gravity offshore storage and loading structures, they appear to be feasible for all base case conditions and could provide storage capacities up to 3 million barrels for the 450' water depths and 2 million barrels for 300' water depth. Some combinations of upper bound water depth, upper bound seismic loading, and large storage requirements may not be feasible on base case cohesive soils. The use of gravity storage strucures on lower bound cohesive soils (St. George or Navarin) is not thought to be feasible; however, it is possible to site the structure remote from the drilling-production platform at a location with suitable soils.

Tanker loading from a rigid single point mooring integral with the gravity storage structure is thought to be feasible. Since this type of tanker loading facility has not been constructed in the past, and since the situation is complicated by sea ice, further work is required to assure feasibility.



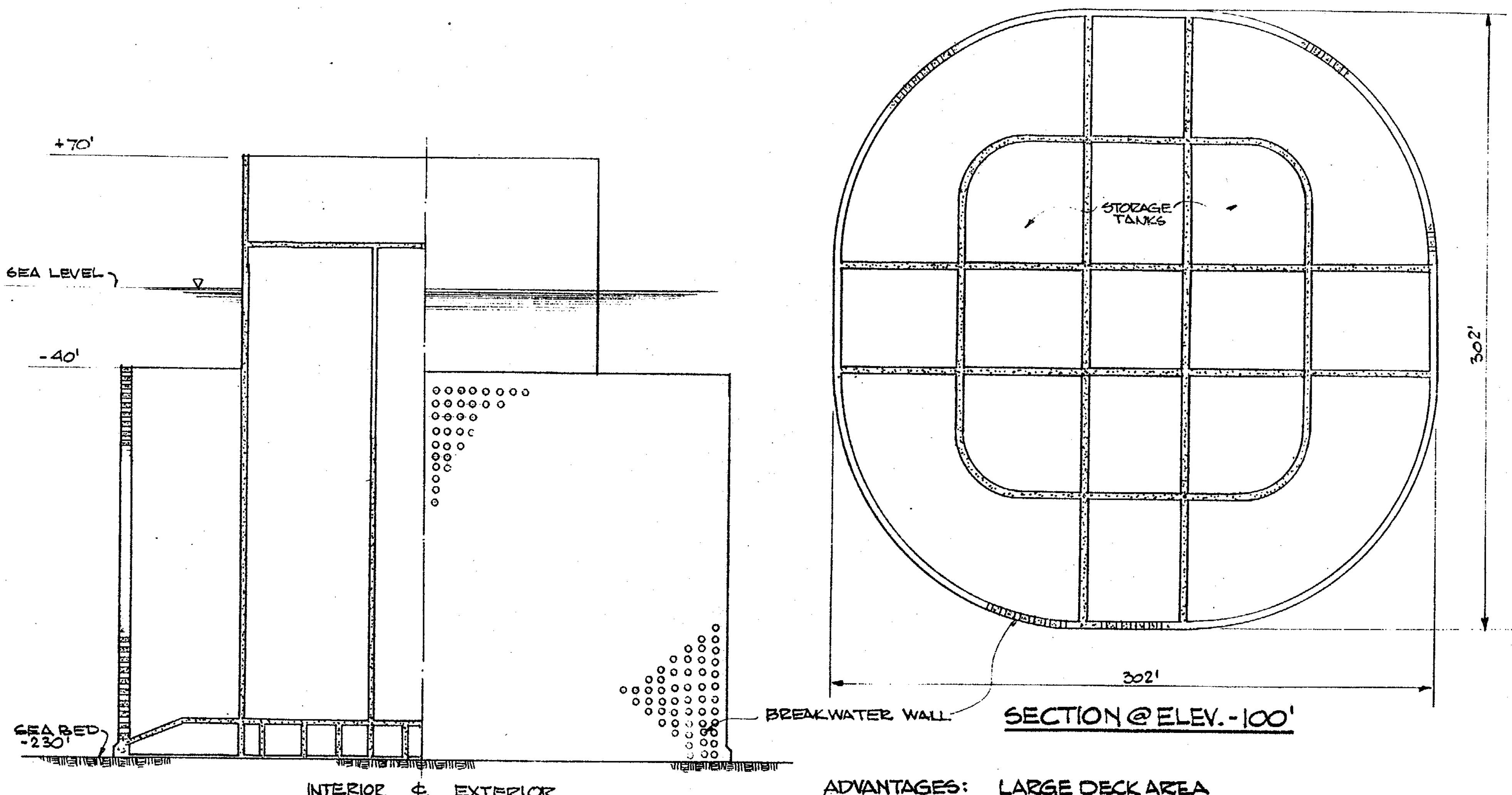
ADVANTAGES: GOOD ICE PERFORMANCE
RIGID MECHANICAL CONNECTIONS

DISADVANTAGES: TRANSPORTATION & INSTALLATION DIFFICULTY.
TANKER OVERRIDE PEDESTAL

ELEVATION

CONDEEP TYPE OIL STORAGE
W/RIGID INTEGRAL SPM.
STORAGE TO 300,000 BBL.

FIGURE 9-1



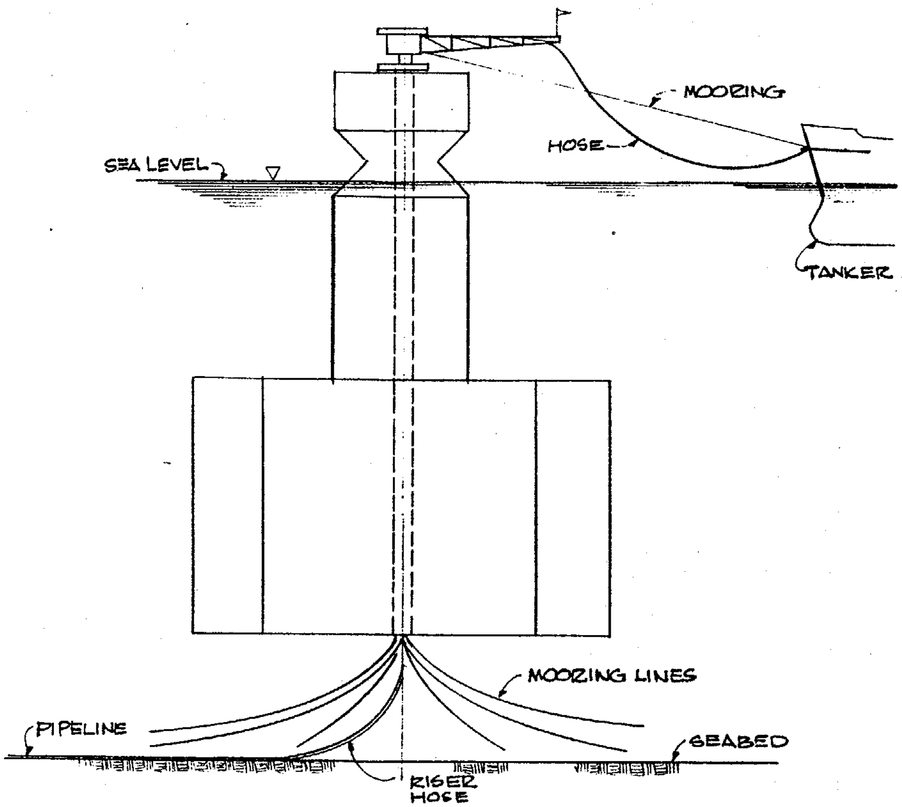
UNDERWATER STORAGE TANK

SECTION @ ELEV. -100'

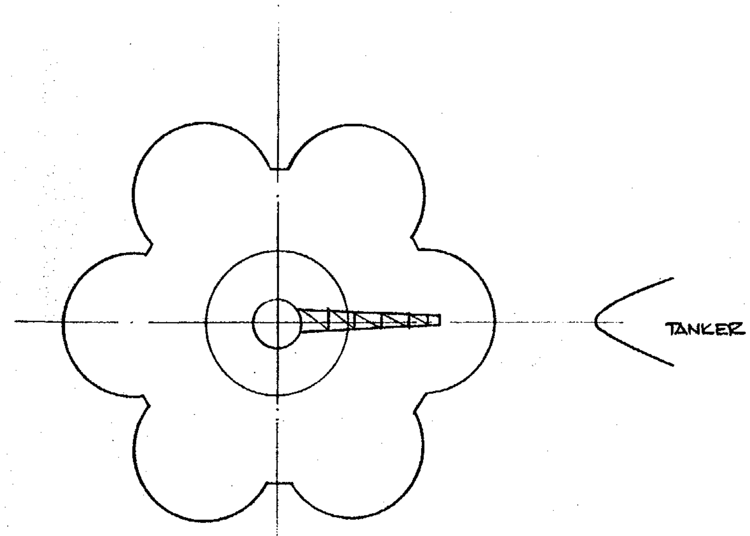
ADVANTAGES: LARGE DECK AREA
GOOD GEOMETRY FOR STORAGE

DISATVANTAGES: LARGE WAVE & ICE FORCES

FIGURE 9-2



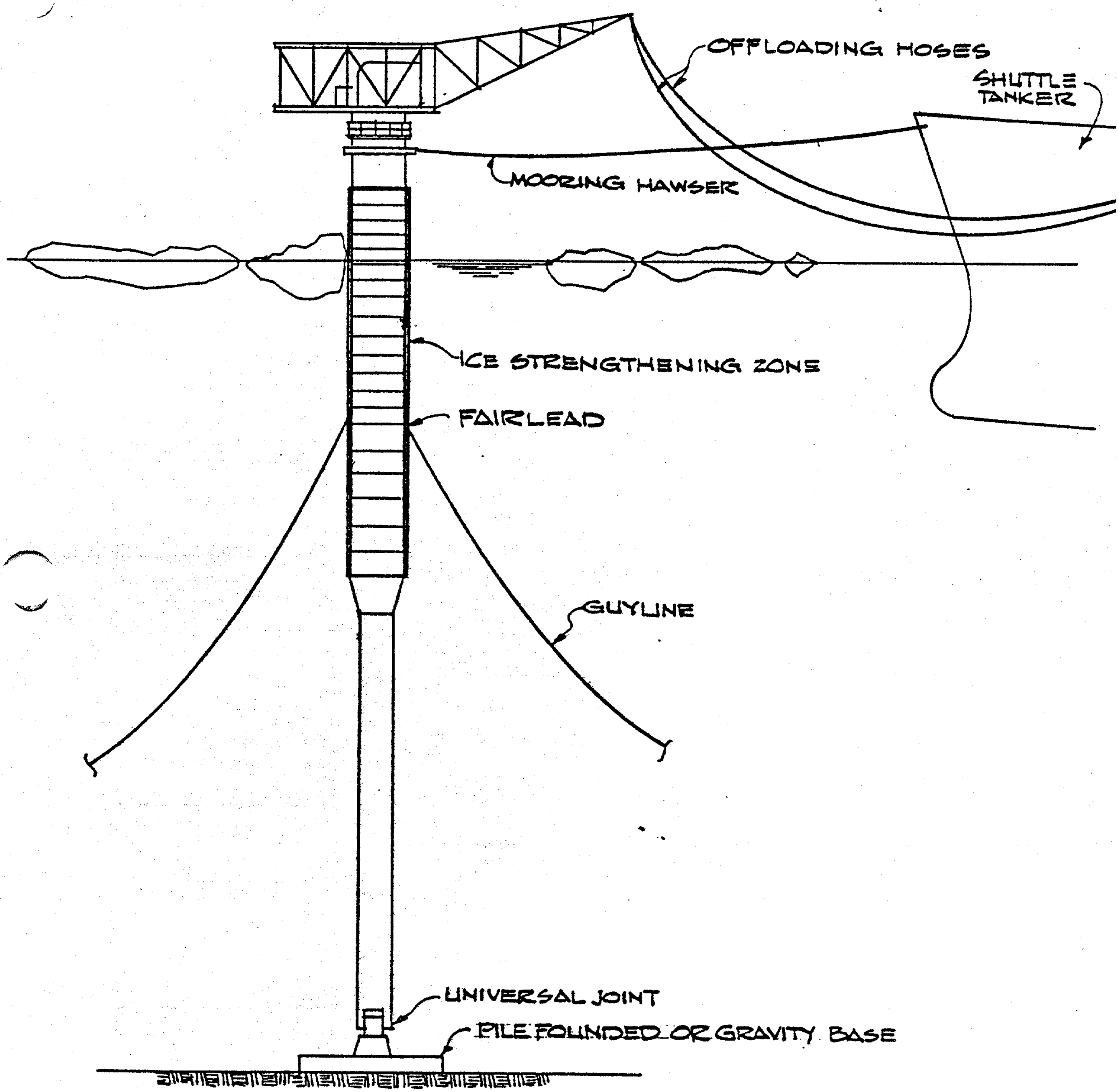
ELEVATION



PLAN

FLOATING CAISSON CONCEPT

FIGURE 9-3



SINGLE POINT MOORING
FOR BERING SEA PRODUCT OFFLOADING

FIGURE 9-4

① Power Generation Module
100' × 45' × 25'
- Two Turbogenerators
- Two Switch Gear Units

② Ballast Water Treatment Module
50' × 50' × 25'
- Oil Water Separator
- Four Units (Induced Air Floation Units - IAF)

③ Hatch 30' × 20'
and Store/Kitchen
Total Dimension 55' × 30' × 25'

④ Control Room/Office and Two bedrooms
55' × 30' × 25'

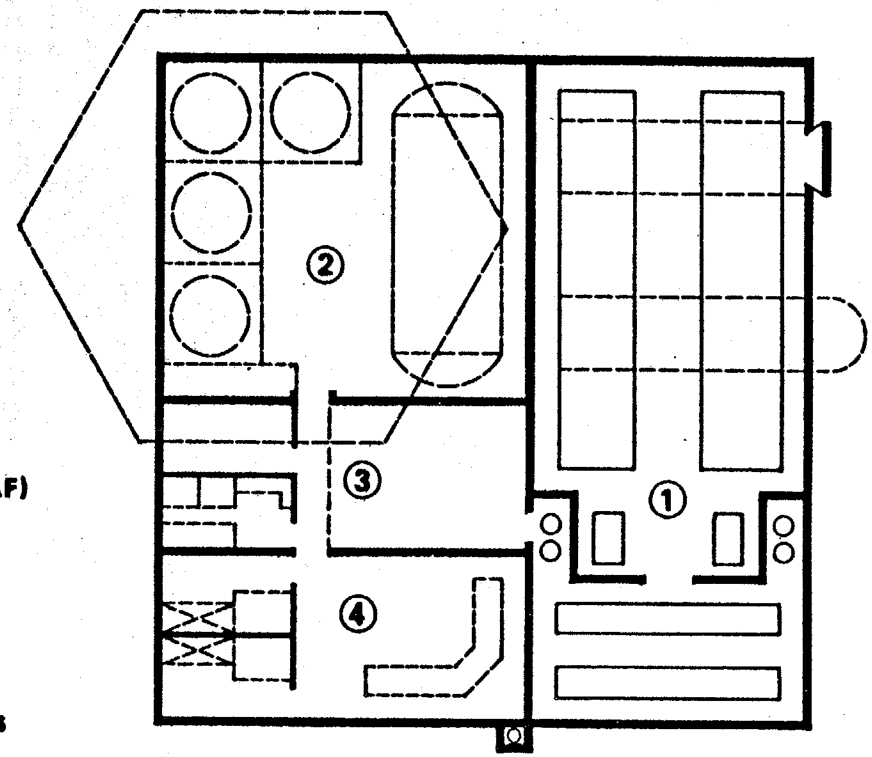
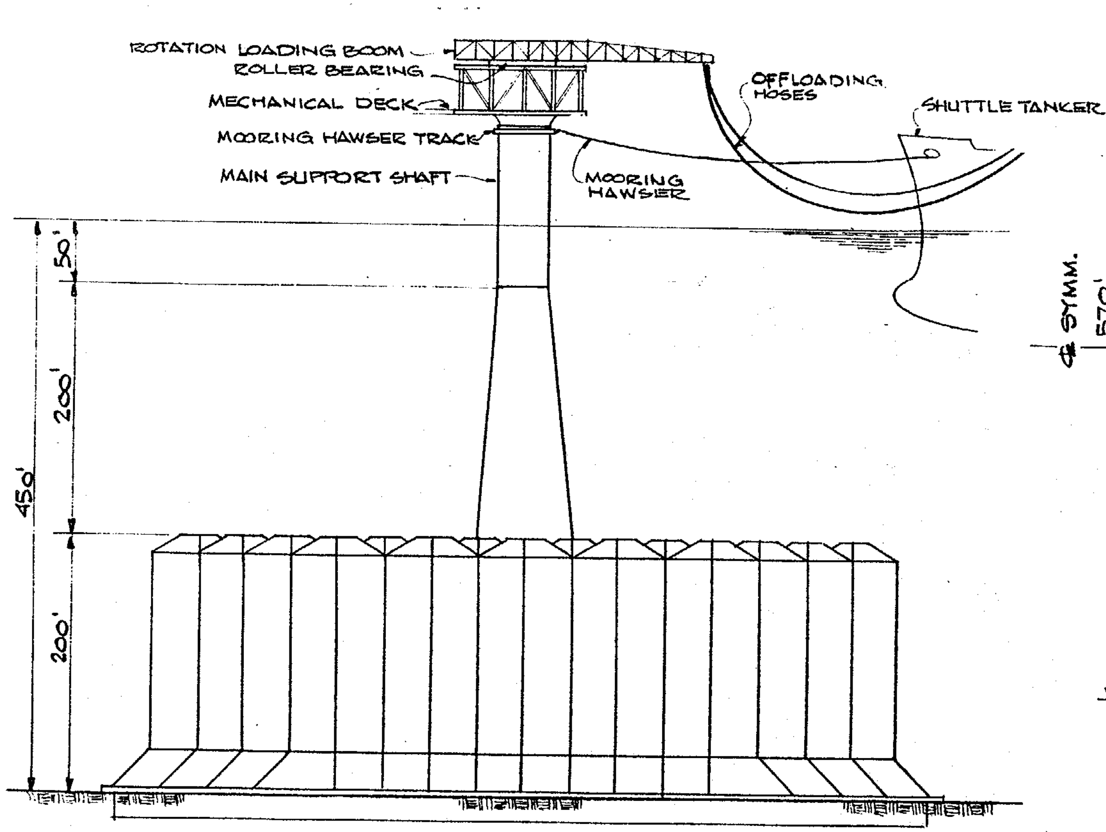


Figure 9 – 5

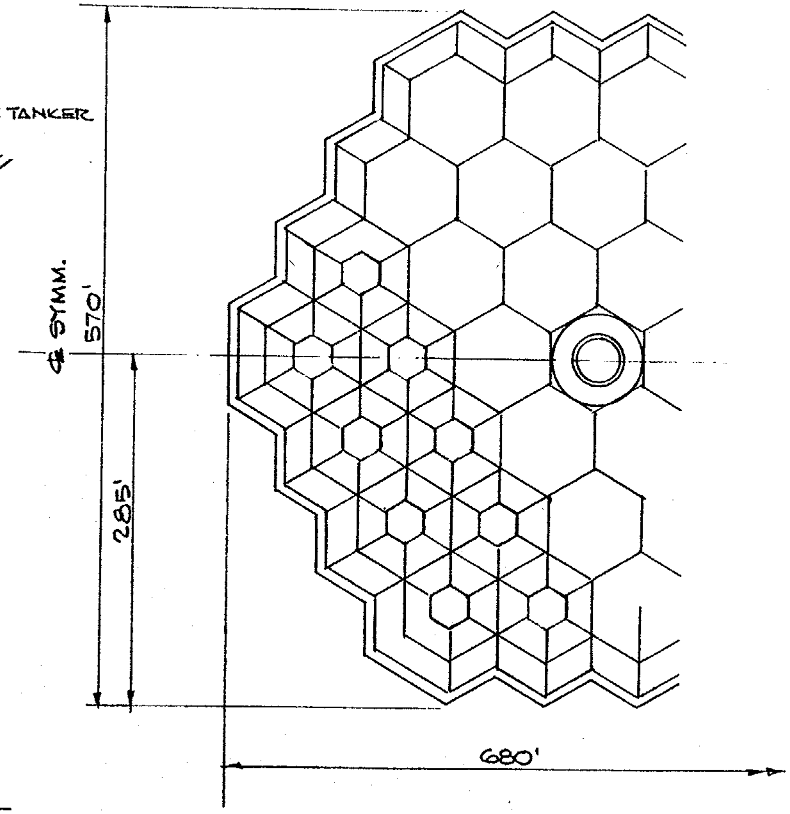
Southern Bering Sea Production Systems Study

OFFSHORE LOADING PLATFORM
TOPSIDES

Job 15154 June 1982



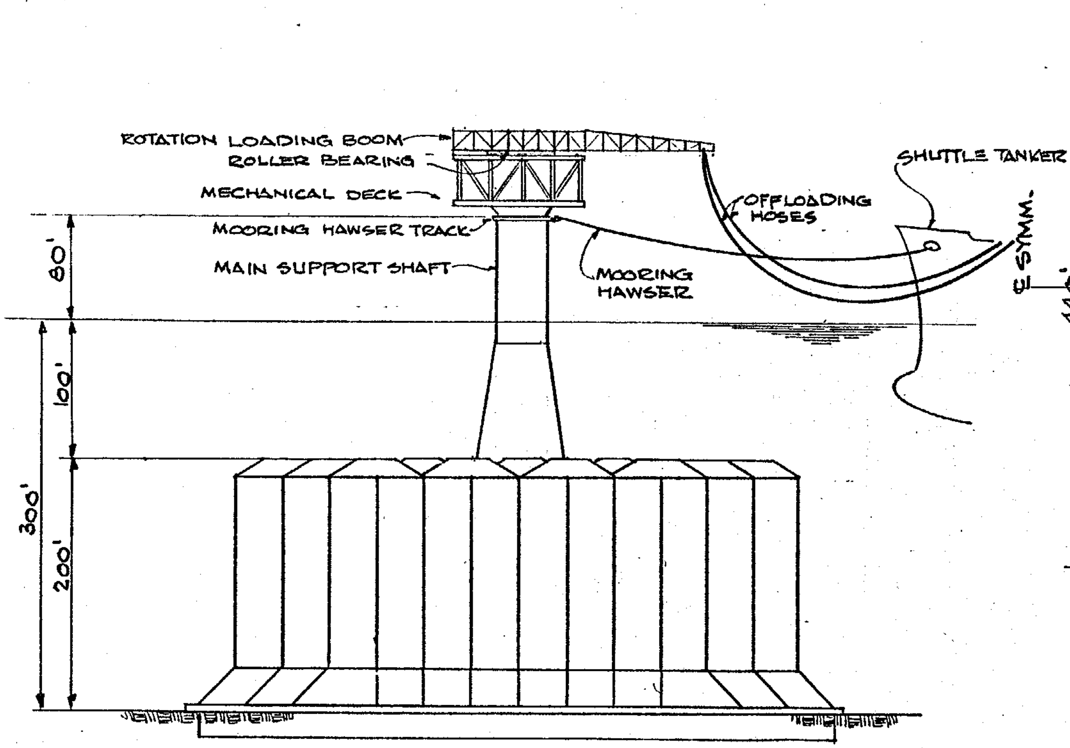
ELEVATION



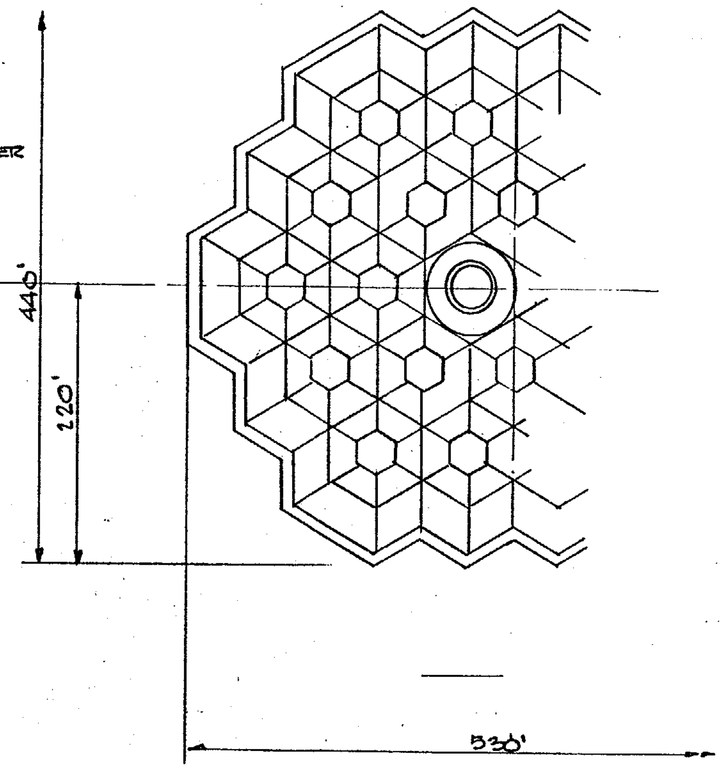
PLAN

CONCRETE STORAGE/LOADING STRUCTURE
WATER DEPTH 450 FT.
STORAGE CAPACITY 3.000 000 BBL

FIGURE 9·6



ELEVATION



PLAN

FIGURE 9-7

CONCRETE STORAGE/LOADING STRUCTURE
WATER DEPTH 300 FT.
STORAGE CAPACITY 2000000 BBL.

TABLE 9-1

MATERIAL TAKE OFF - 450' W.D.

A. 1.2×10^6 BBL[1]

Base Slab	54409 CY
Enlargement at Base-Skirt Connection	1000 CY
Interior Cell Walls	69924 CY
Enlargement at Base-Wall Connection	1000 CY
Exterior Cell Walls	14107 CY
Exterior Cell Walls	9069 CY
Roof of Cells	18734 CY
Tower[2]	10334 CY
Apron	5000 CY
Ring Beam	1000 CY
Miscellaneous	10000 CY
Total	194577 CY

B. 3×10^6 BBL[1]

Base Slab	54409 CY
Enlargement at Base-Skirt Connection	1000 CY
Interior Cell Walls	139848 CY
Enlargement at Base-Wall Connection	1000 CY
Exterior Cell Walls	28214 CY
Exterior Cell Walls	18188 CY
Roof of Cells	18734 CY
Tower[2]	7844 CY
Apron	5000 CY
Ring Beam	1000 CY
Miscellaneous	10000 CY
Total	285187 CY

Notes:

[1]Both structures assumed to be located at the design basis water depth (450 feet).

[2]Assumed height above mean sea level 65 feet.

TABLE 9-2

MATERIAL TAKE OFF - 300' W.D.

A. $1x10^6$ BBL[1]

Base Slab	32835 CY
Enlargement at Base-Skirt Connection	1000 CY
Interior Cell Walls	41056 CY
Enlargement at Base-Wall Connection	1000 CY
Exterior Cell Walls	8061 CY
Exterior Cell Walls	9069 CY
Roof of Cells	9407 CY
Tower[2]	4440 CY
Apron	5000 CY
Ring Beam	1000 CY
Miscellaneous	10000 CY
Total	121868 CY

B. $2x10^6$ BBL[1]

Base Slab	32835 CY
Enlargement at Base-Skirt Connection	1000 CY
Interior Cell Walls	82112 CY
Enlargement at Base-Wall Connection	1000 CY
Exterior Cell Walls	16122 CY
Exterior Cell Walls	18138 CY
Roof of Cells	9407 CY
Tower[2]	2765 CY
Apron	5000 CY
Ring Beam	1000 CY
Miscellaneous	10000 CY
Total	179379 CY

Notes:

[1]Both structures assumed to be located at the design basis water depth (300 feet).

[2]Assumed height above mean sea level 65 feet.

TABLE OF CONTENTS

SECTION 10.0

Page

LI: ' OF FIGURES

SI :TION 10.0

Tit e

LIST OF TABLES

SECTION 10.0

10.0 COST ESTIMATES AND SCHEDULES - TRANSPORTATION SYSTEMS

10.1 General

10.1.1 Scope of Work

The cost estimates presented in this section are based on the preliminary designs which are presented in this report. Included are cost estimates for the following items:

- Pipelines from All Basins to a Shore Terminal
- Shore Terminal
- Offshore Storage and Loading Structure

Note: Costs for steel and concrete structures and topsides are given in Section 6.

10.1.2 Objectives

This cost estimate has been presented in such a way that the following objectives will be met as much as possible within the limited framework of the project.

- The assumptions or background sources of information are clearly identified.

- The cost estimating methods and basis of each major cost item is identified.

- This data base will provide a means for future cost estimates, tailored to the requirements of individual operating companies.

10.1.3 Date of Estimate and Escalation

The estimated costs are based on June 1982 costs in U. S. Dollars. No escalation beyond this time has been included.

10.1.4 Accuracy of Estimate and Cost Estimating Allowance

The cost estimates included herein are thought to be accurate to within plus or minus 20 to 30 percent. A cost estimating allowance of 30 percent has been included in all of the cost summary sheets in this section. It is identified on each summary work sheet as such. It is recommended that this allowance be carefully reviewed to ensure that it is consistent with the level of confidence assigned to both the design basis conditions and the cost estimating bases and methods.

10.1.5 Home Office Costs

The summary tables include an allowance of 10 percent for home office costs. This allowance is intended to cover the following items:

- Design engineering
- Project management
- Site investigations
- Purchasing and expediting

10.2 Pipelines

10.2.1 Items Included

Six pipeline systems are cost estimated, one 24" O.D. pipeline and one 36" O.D. pipeline each from the North Aleutian, The St. George and the Navarin Basins to Lefthand Bay on the Alaska Peninsula. An intermediate booster station is included for the two pipeline systems cost-estimated for the Navarin Basin.

The estimates include all pipeline materials and freight costs, construction costs and field survey costs based on the the schedule and layfleet requirements presented earlier in this report.

Mainline pumps on the production platform are not included here. They are treated as part of the platform topsides costs.

10.2.2 Basis of Cost Estimate

10.2.2.1 Pipeline Materials

The pipeline steel cost is estimated at $700 per ton.

The cost of weight coating, corrosion coating, anodes and buckle arrestors can be expressed in terms of the unit weight of pipe:

- $367 per ton of 24" O.D. pipe
- $329 per ton of 36" O.D. pipe

10.2.2.2 Ocean Freight

Ocean freight is estimated at $100 per pay ton.

10.2.2.3 Labor

Contractor's labor for pipelines is estimated at $75 per hour offshore including allowance for catering and contractor's overhead and profit. Contractor's labor onshore is estimated at $156 per hour including camp and catering (see Section 10.3.2.2).

10.2.2.4 Marine Survey

An allowance is made for marine surveys of the pipeline routes as follows:

- North Aleutian Basin to Herendeen Bay - $3,000,000
- St. George Basin to Herendeen Bay - $7,000,000
- Navarin Basin to Herendeen Bay - $10,000,000

10.2.2.5 Pipeline Shore Connections

An allowance of $60,000,000 is made for all onshore activities related to construction of the pipeline shore landings. This includes equipment, materials and labor for the construction of sheet-pile and trestle structures, surf zone trenching, construction of onshore pipeways, welding of pipe lengths onshore, pipe buoyancy elements, pipe-pull winch capital costs, foundation work and freight costs.

Offshore work associated with the pipeline landings is done by a laybarge, and this cost is included in the barge day rates and the time allowance for effecting the work.

10.2.2.6 Laybarge Mobilization Costs

Laybarges of the 2nd generation type are assumed to be mobilized from the Gulf of Mexico (through the Panama Canal) or from the Western Pacific or from Southeast Asia. An allowance of $19,000,000 for mobilization and demobilization of each 2nd generation laybarge is assumed.

The 3rd generation barges with semi-submersible hulls needed for pipeline construction to the Navarin platform are both few in number and potentially expensive to mobilize and demobilize from Europe to the Bering Sea. Primarily because of the questionable availability, it is assumed that two of them would be constructed in Japan or Korea with an initial capital outlay of $160,000,000 each. Of this amount, $50,000,000 each is deducted from the total cost accumulation in barge day rates during the two seasons of work planned for these barges. This reduction is due to operator's ownership of the barges which precludes contingency and profits.

No account is taken of return on investment through sale or lease of the barges at the conclusion of the construction.

10.2.2.7 Laybarge Day Rates

Total operating costs for laybarges were obtained as follows:

	2nd Gen. Barge	3rd Gen. Barge
Bare barge cost incl. fuel, consumables and insurance (est.)	$140,000/day	$212,000/day
Labor	225,000/day(1)	$315,000/day(2)
Total day rate	$365,000/day	$527,000/day
Day rate reduction	---	$136,000/day(3)
Net day rate	$365,000/day	$391,000/day

Notes:

(1) 250 men @ $75 per mh
- 12 hr. working day, 7 days a week

(2) 350 men @ $75 per mh
- 12 hr. working day, 7 days a week

(3) $50,000,000 return over two working seasons (183 days per season)

10.2.2.8 Submarine Tenders and Submarines

Submarines and their tender used for survey and inspection during the pipelay are assigned a day rate of $60,000. The number of these vessels varies with the number of barges required for each pipe size and length. Mobilization and demobilization of each tender is assumed to be $1,000,000.

10.2.2.9 Work Barges

Work barges are assumed to be used for pipeline tie-ins, repairs and hydrostatic testing. An allowance of $3,000,000 is made per pipeline tie-in and $4,000,000 per pipeline repair, regardless of pipe diameter. The number of estimated tie-ins and repairs are tabulated below.

TIE-INS AND REPAIRS

	24" O.D. Pipe		36" O.D. Pipe	
	Tie-Ins	Repairs	Tie-ins	Repairs
N. Aleutian	2	1	2	1
St. George	8	2	10	2
Navarin	6	4	8	4

Mobilization and demobilization costs are assumed to be $12,000,000 per work barge.

10.2.2.10 Pipeline Trenching

In all cases of pipeline size and route combinations, it is assumed that 20 nautical miles of trenching from shore into the Bering Sea will be sufficient to protect the pipe and to meet regulatory requirements. An allowance of $46,000,000 is made for mob/demob of the trenching barge and sled and for the estimated 80 days of work required.

10.2.2.11 Dredging

A small dredge will be required to prepare the seabed slope at the cargo dock in Lefthand Bay and to assist in the cut through Deer Island in Herendeen Bay. A sidecaster would be suitable for these purposes.

Assuming the dredger can be mobilized from Alaskan waters, an allowance of $2,000,000 is made for this work including mobilization and demobilization.

10.2.2.12 Laybarge Support Fleet

Each laybarge has an attendant fleet of two anchor handling tugs and a survey vessel. Pipehaulers are used in sufficient number to meet anticipated barge lay rates.

Total estimated day rates for the barge fleet support are $72,000/day for a 2nd generation barge and $144,000/day for a 3rd generation barge. An allowance is made for additional pipehaulers in the case of a 36" O.D. line from the Navarin Basin to Herendeen Bay in the amount of $7,000,000.

10.2.2.13 Laybarge Winter Layover

An allowance has been made for winter layover between the two summer construction seasons. This day rate allowance covers maintenance and rental of the inactive barges.

- 2nd generation barge: $ 20,000 per day
- 3rd generation barge: $100,000 per day

10.2.2.14 Booster Station

The costs of the booster station on the pipelines to the Navarin Basin are given in Table 10 - 1.

10.2.2.15 Onshore Pipeline

The total cost estimated for the onshore segment of pipeline including labor equipment and materials results in the a unit cost of $21 per dia. in. per ft.

10.2.3 Order of Magnitude Capital Cost Estimates

The capital cost estimates are given in Tables 10-2 through 10-8.

10.2.4 Cash Flow

Cash flow during construction is plotted in Figures 10 - 1 through 10-6 for each case of pipe size and platform location. The histograms are typically bimodal showing work concentrated in two summer seasons.

Cash flow for the Navarin Basin construction shows three peaks, the first being for capital commitment to two, 3rd generation laybarges before the first season of work at sea.

10.2.5 Operating Costs

Most of the annual operating expenses for a pipeline usually result from fuel costs for the pumps and maintenance costs for the pump stations. In the cases of the North Aleutian and St. George pipelines, these costs are absorbed into the operating expenses for the platform topsides.

Other operating expenses result from:

- internal inspection and pigging
- external inspection (annually)
- maintenance and repair
- pipeline communications

10.2.5.1 Internal Inspection and Pigging

Normal pigging operations using caliper pigs for internal inspection of the pipeline or scraper pigs are the responsibility of the platform crews for pig launching and of the shore-terminal personnel for receiving. No separate operating expense is identified as part of the pipeline operating costs.

10.2.5.2 External Inspection

An annual survey and inspection of the entire exposed part of the pipeline is required. This survey can be done with a manned submersible or an unmanned vehicle tethered to a surface vessel. If visual inspection of the pipeline for damage to the concrete coating is not required, part or all of this survey can be done by a surface vessel towing a side-scan or scanning sonar.

The purpose of the survey is to disclose:

- concrete or pipe damage from trawl-boards or anchors
- anodes needing replacement

- unsupported pipe spans
- changes in the seabed support of the pipe or pipe exposure due to waves, currents or seabed liquefaction

Assuming a manned submersible is used for the inspection and that the submersible and its tender are mobilized out of Seattle, the annual survey of the 26 nautical miles of North Aleutian pipeline can probably be done for about $500,000. The survey of the St. George pipeline using this same equipment is estimated to be about $2,500,000 and for the Navarin pipeline, about $6,500,000.

10.2.5.3 Maintenance and Repair

Minor repairs and maintenance consist of:

- sandbagging localized areas of seabed scour
- pipe support at free spans
- anode replacement

These can be carried out by a diving spread operating from a work vessel or supply boat. Estimate about $500,000 a year for such maintenance work.

A major repair activity is required when the line is dented or buckled by an anchor or a pipe sag at the support-point of a free span. A section of the line must be cut out and replaced by a pup using either mechanical connectors or hyperbaric welding. A figure of $4,000,000 was used as the estimate of a pipeline repair during construction. Add an additional $1,000,000 for mobilization and demobilization of the work fleet and equipment for a total of $5,000,000 per major pipeline repair.

10.2.5.4 Communications

If the ALASCOM satellite system is used for pipeline communications, there is a charge of $9,800 per month for operating and maintenance costs for each earth station plus a monthly tariff charge. For the St. George and Navarin Basins the estimated annual cost for this service is estimated below (the North Aleutian platform is assumed to be serviced by a microwave system).

Platform Location	Annual Pipeline Communication Expense
St. George	$250,000
Navarin	$390,000

10.3 Shore Terminal and Resupply Base

10.3.1 Items Included

The terminal for crude oil storage and tanker loading, the resupply base and the airstrip are treated as one complex with sharing of some services among them. Major components of these installations are shown in tables and layouts of the previous text.

The facilities complex is a grass roots development, and all costs of site preparation and equipment mobilization are included in this estimate.

It is assumed that:

- There is an adequate fresh water supply from an existing spring.
- An adequate quarry for rock/gravel is near the site locations.

Items excluded from this estimate are:

- Cost of land acquisitions
- Cost of environmental impact report and permits
- Cost of construction and operating permits
- Owners' cost, financing and interest during construction, insurance
- Prestart-up and start-up costs
- Spare parts
- Escalation beyond the 1st and 2nd Quarter of 1982

10.3.2 Basis of Cost Estimate

10.3.2.1 Equipment, Materials and Site Preparation

- Storage tank costs are estimated on the basis of dollars per barrel capacity for complete installations.
- Process unit costs are factored from historical data for similar jobs and adjusted to reflect the proper capacity and current cost.
- Loading area and marine facilities costs are also factored from similar jobs and include adjustments for size and conditions at the location.
- On-site power generation and electrical distribution-system costs are estimated from megawatt requirements and include transmission line costs to the resupply base.
- Site preparation and improvements, off-plot and buildings' costs are developed from the conceptual plot plan using rough earthwork quantities, plot and building areas as pricing parameters.

10.3.2.2 Labor

- A manual labor wage rate of $50 per manhour has been used. Labor productivity is based on experience in Alaska and Canada.
- The following labor-related costs are included:

- Camp/catering at $20 per manhour
- Field indirects* at 140% of direct labor cost
- Construction management is estimated at 3% of total labor, material and subcontract costs.

* Field indirects include temporary construction facilities, construction equipment, tools and consumables, miscellaneous construction services and field office costs.

10.3.2.3 Alaska Use Tax

An Alaska use tax of 1% of total field costs (management, labor, materials, subcontract) is added for work done onshore.

10.3.3 Order of Magnitude Capital Cost Estimates

The capital cost estimates for onshore facilities including 10% home office costs and 30% contingency are given in Table 10-9 .

10.3.4 Cash Flow

Cash flow diagrams for the onshore terminal facilities for 100 MBPD and 300 MBPD cases of production throughput are given in Figures 10-7 and 10-8 .

10.3.5 Operating Costs

10.3.5.1 Storage and Loading Terminal

Annual operating expenses of the terminal complex at Lefthand Bay are expected to be about 3.5% of the total capital cost. This includes:

- wages
- catering
- fuel
- local transportation and air travel
- maintenance and repair
- insurance

On this basis, the yearly terminal operating expenses for the 100 MBPD case of production throughput is expected to be about \$20,000,000 and for the 300 MBPD case, \$27,000,000.

10.3.5.2 Resupply Base

The resupply base operating expenses cover not only the permanent employees' wages, housing and cargo handling equipment, but also the supply-boat maintenance and fuel expenses. Assuming a long-term lease rate for ice-breaking supply boats of \$6000 per day each, exclusive of fuel, the following table summarizes the expected annual supply boat operating expenses for servicing each of the lease basins.

	North Aleutian	St. George	Navarin
No. of supply boats	3	5	8
Yearly lease cost ($\$ \times 10^6$)	6.6	11	18
Fuel Consumption (bbl $\times 10^3$)	30	120	360

Assume the diesel fuel produced at the topping plant is valued at $0.40 per gallon and that operating expenses of the resupply base, apart from supply boat operations, are about $6,000,000. Then the total expected operating costs are summarized in the following table.

Resupply Base Annual Operating Expenses
(U.S. Dollars x 10^6)

	North Aleutian	St. George	Navarin
Onshore facilities	6	6	6
Supply boat lease	7	11	18
Supply boat fuel	0.5	2	6
Totals	14	19	30

10.4 Offshore Storage and Loading Structure

10.4.1 Items Included

This estimate includes all material equipment and labor cost associated with fabrication, transportation, and installation of the storage/loading structure for two storage volumes in each of the three basins.

10.4.2 Basis of Cost Estimate

The costs included for fabrication of the structure were on the same basis as the concrete drilling and production platform. Items have been factored on the basis of experience of the project team from the drilling and production platforms.

Costs for topside facilities, mooring systems, transfer boom system and transportation and installation are purely allowances.

10.4.3 Cost Summary

Table 10-10 gives the total cost for the storage/loading structures for all basins. These costs contain home office an contingency as discussed in Section 10.1.5.

10.4.4 Cash Flow

Estimated cash flow by quarters is shown in Figures 10-10 and 10-11 for St. George/Navarin Storage and Loading structures (3 million barrels) and North Aleutian structure (2 million barrels).

10.5 Schedules

A master schedule for transportation systems is shown in Figure 10-12. Both the pipeline/shore terminal and the offshore storage and loading schedules are indicated. Even though the completion time of these systems is comparable to the drilling and production structures, they are not on the critical path due to the time taken for drilling and completing the wells.

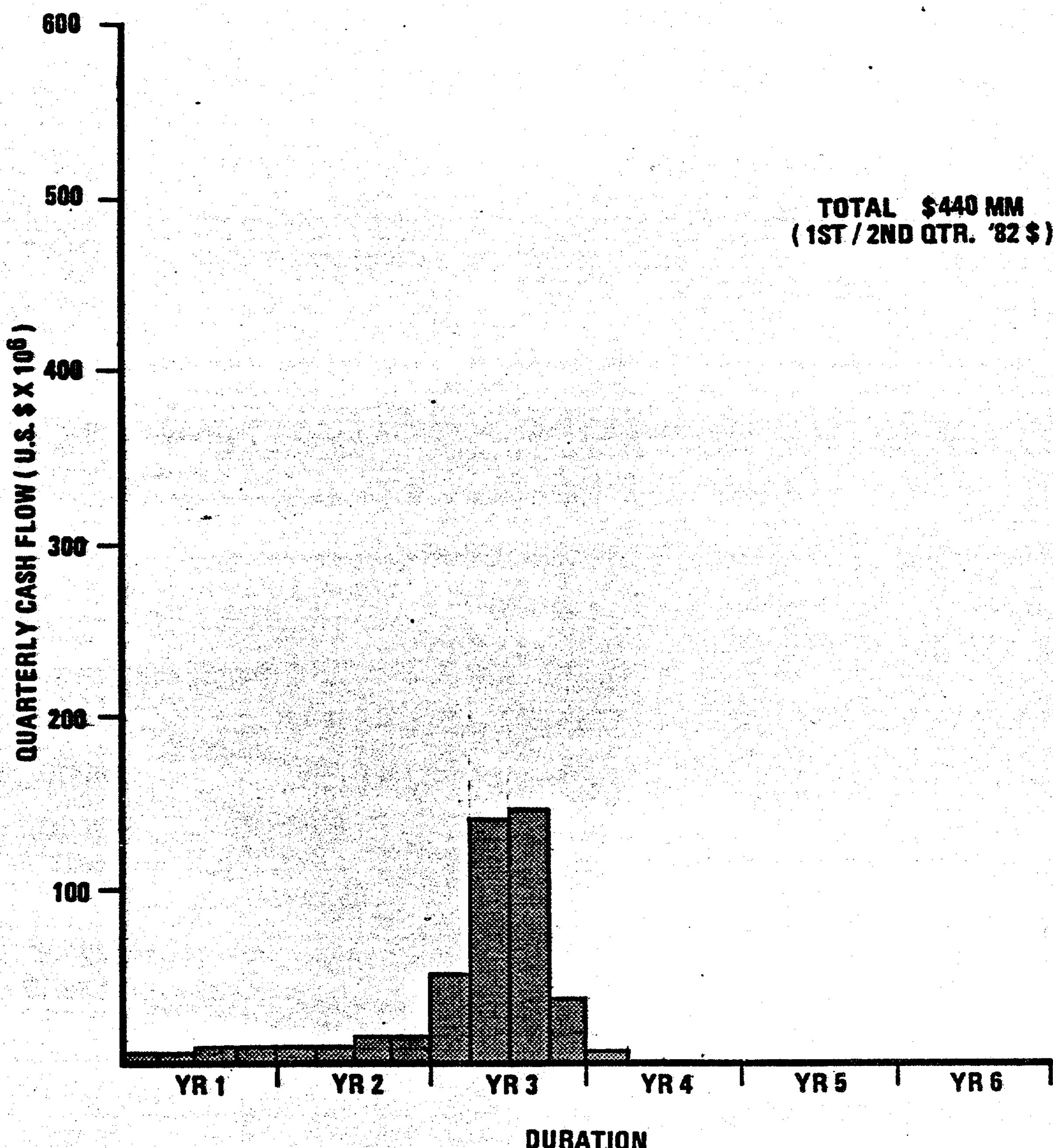


Figure 10-1

Southern Bering Sea Production Systems Study

CASH FLOW
24" O.D. PIPELINE:
N. ALEUTIAN BASIN –
LEFTHAND BAY

Job 15154 BECHTEL June 1982

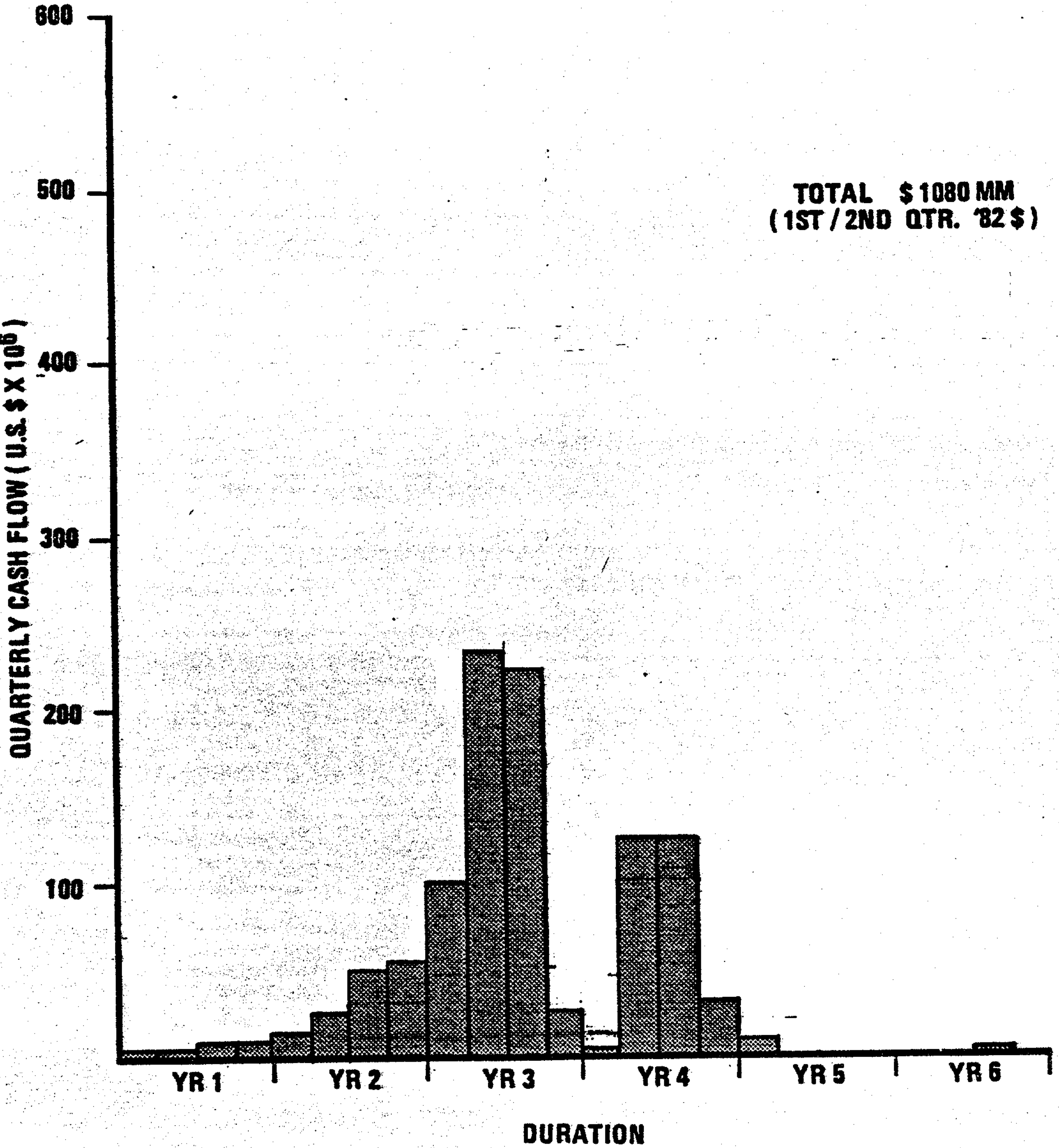


Figure 10-2

Southern Bering Sea Production Systems Study

CASH FLOW
24" O.D. PIPELINE:
ST. GEORGE BASIN –
LEFTHAND BAY

Job 15154 BECHTEL June 1982

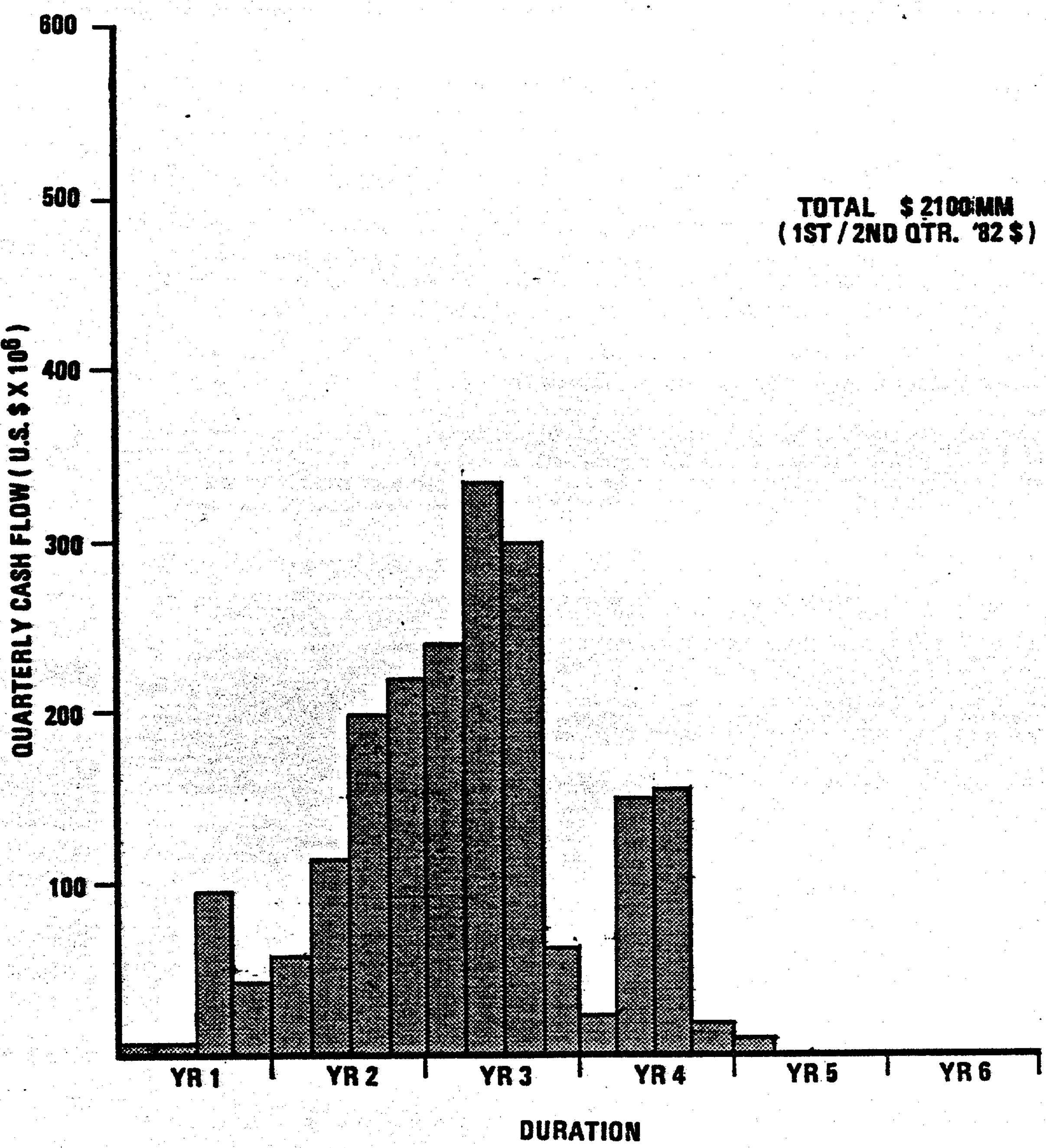


Figure 10-3

Southern Bering Sea Production Systems Study

CASH FLOW
24" O.D. PIPELINE:
NAVARIN BASIN –
LEFTHAND BAY

Job 15154 BECHTEL June 1982

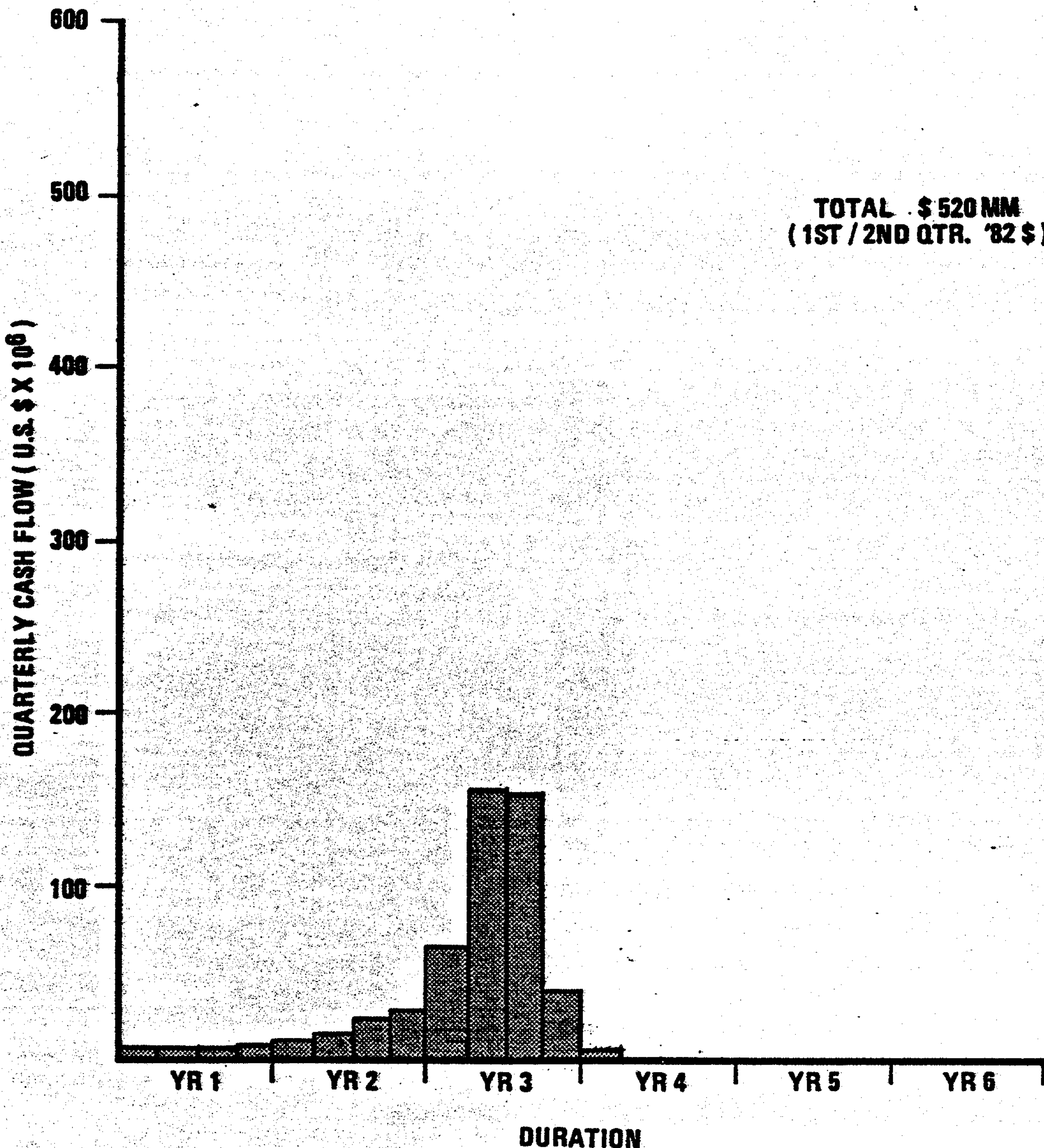


Figure 10–4

Southern Bering Sea Production Systems Study

CASH FLOW
36" O.D. PIPELINE:
N. ALEUTIAN BASIN –
LEFTHAND BAY

Job 15154 BECHTEL June 1982

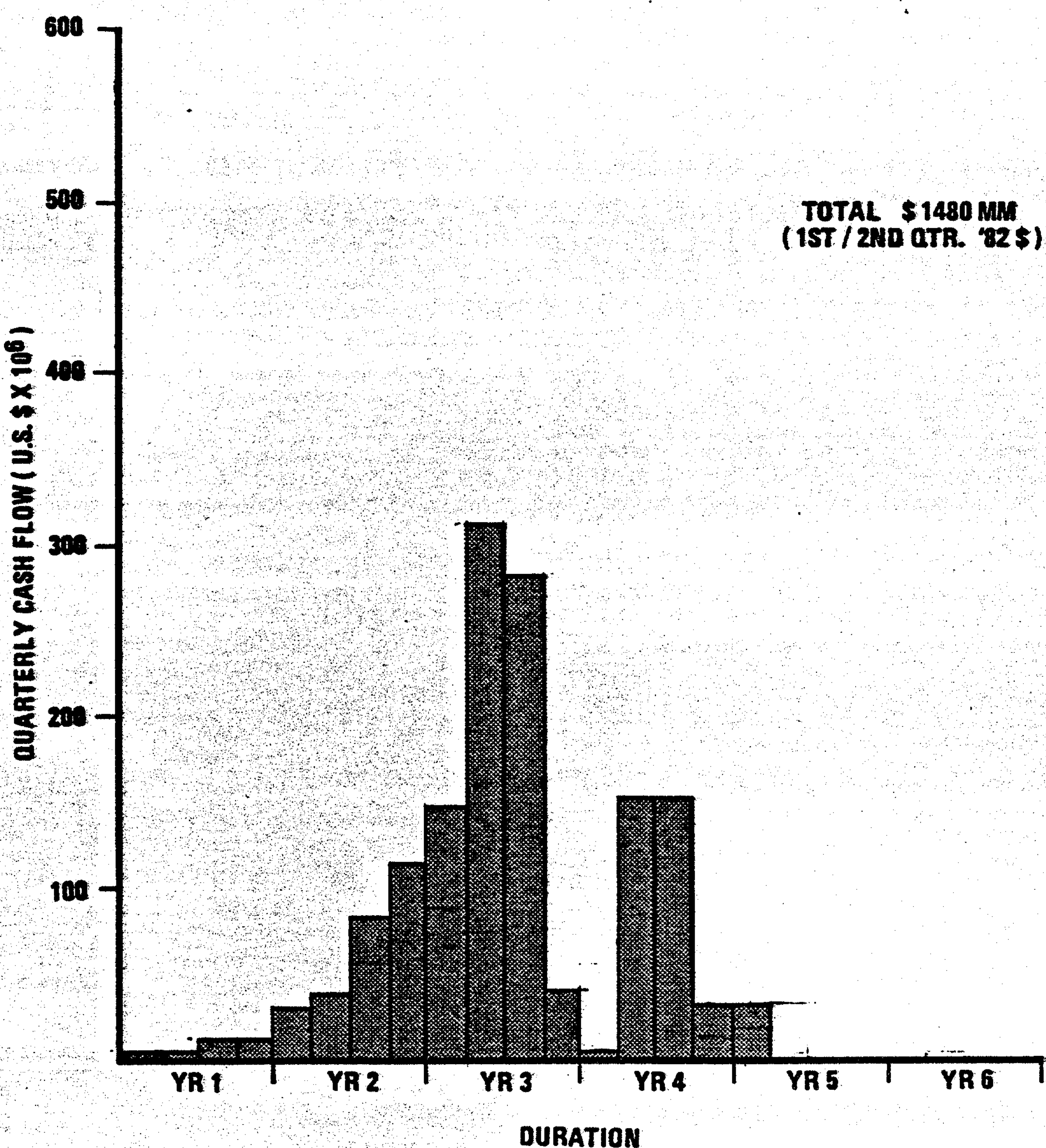


Figure 10–5
Southern Bering Sea Production Systems Study

CASH FLOW
36" O.D. PIPELINE:
ST. GEORGE BASIN –
LEFTHAND BAY

Job 15154 BECHTEL June 1982

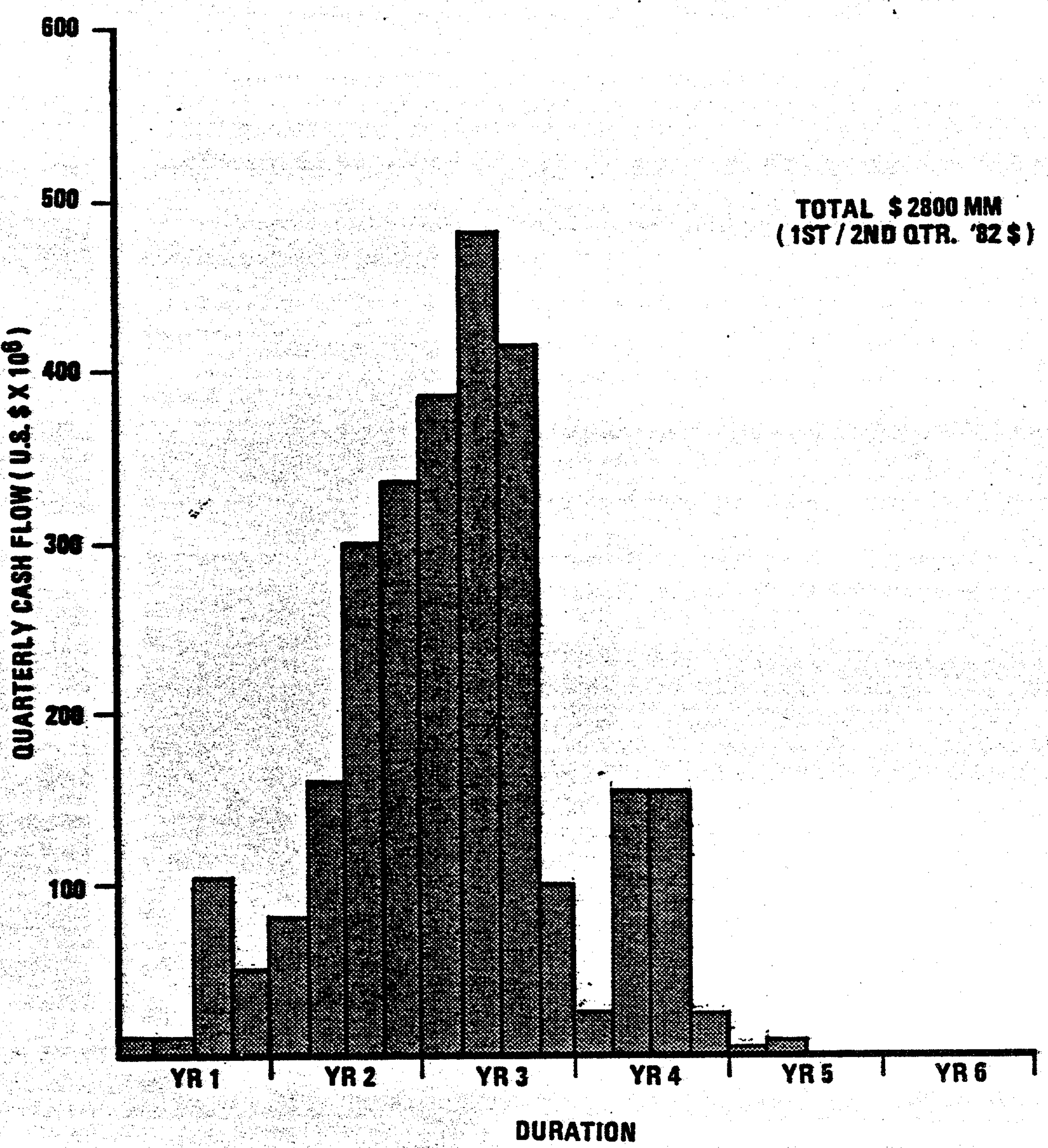


Figure 10-6

Southern Bering Sea Production Systems Study

CASH FLOW
36" O.D. PIPELINE:
NAVARIN BASIN —
LEFTHAND BAY

Job 15154 — BECHTEL — June 1982

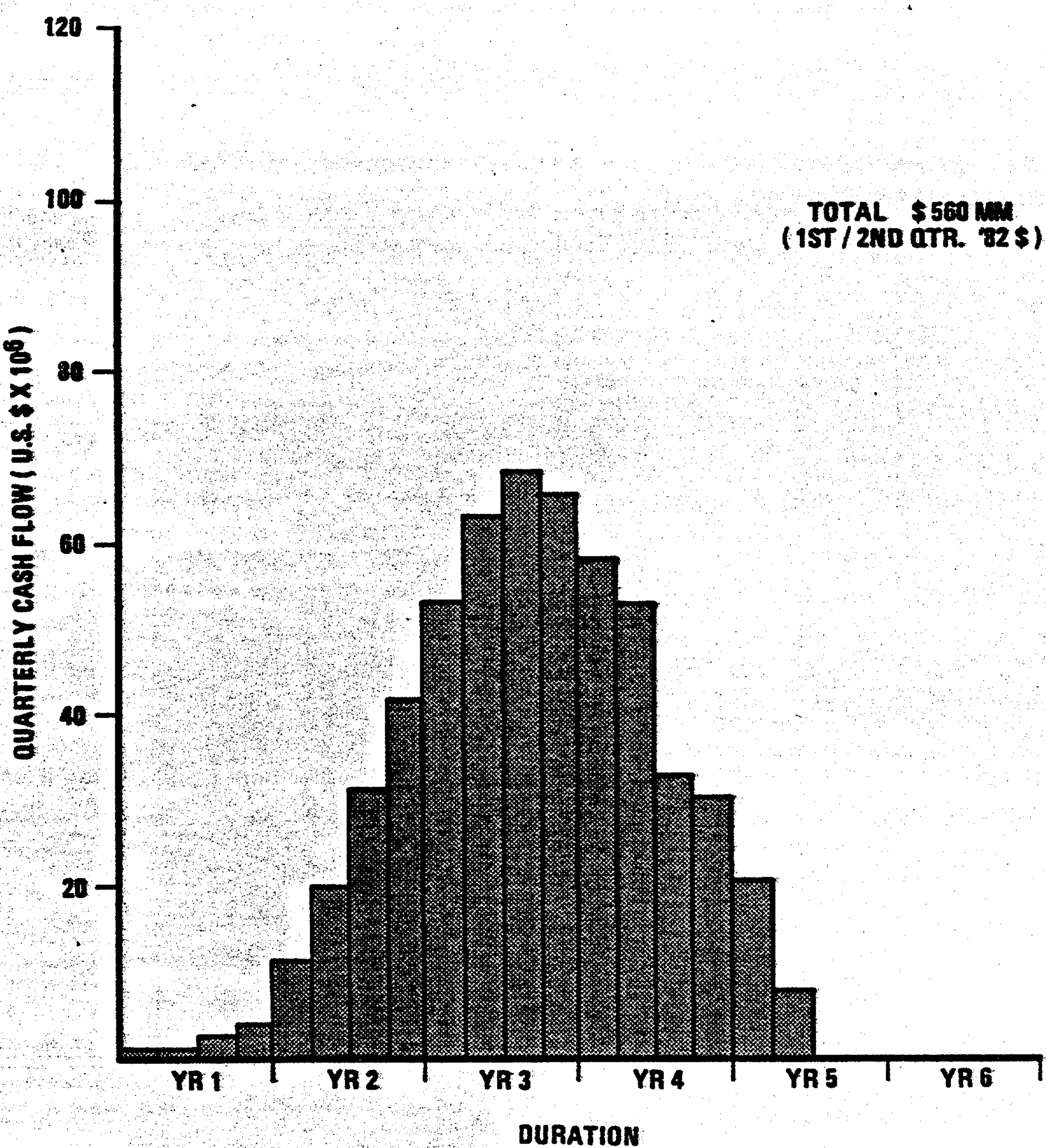


Figure 10-7

Southern Bering Sea Production Systems Study

CASH FLOW
ONSHORE FACILITIES
STORAGE & TERMINAL
(100 MBPD)

Job 15154 BECHTEL June 1982

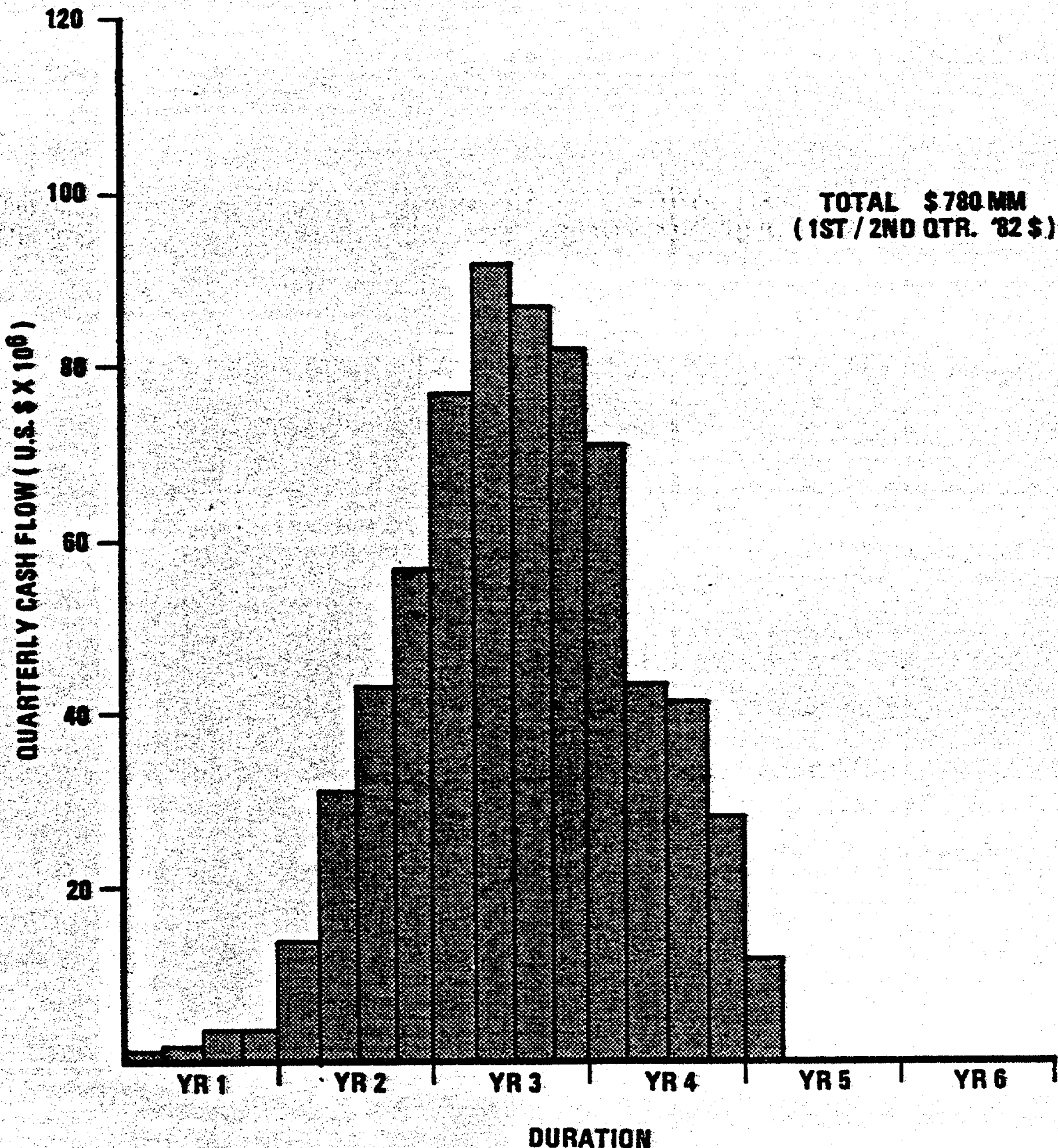


Figure 10-6

Southern Bering Sea Production Systems Study

CASH FLOW
ONSHORE FACILITIES
STORAGE & TERMINAL
(300 MBPD)

Job 15154 BECHTEL June 1982

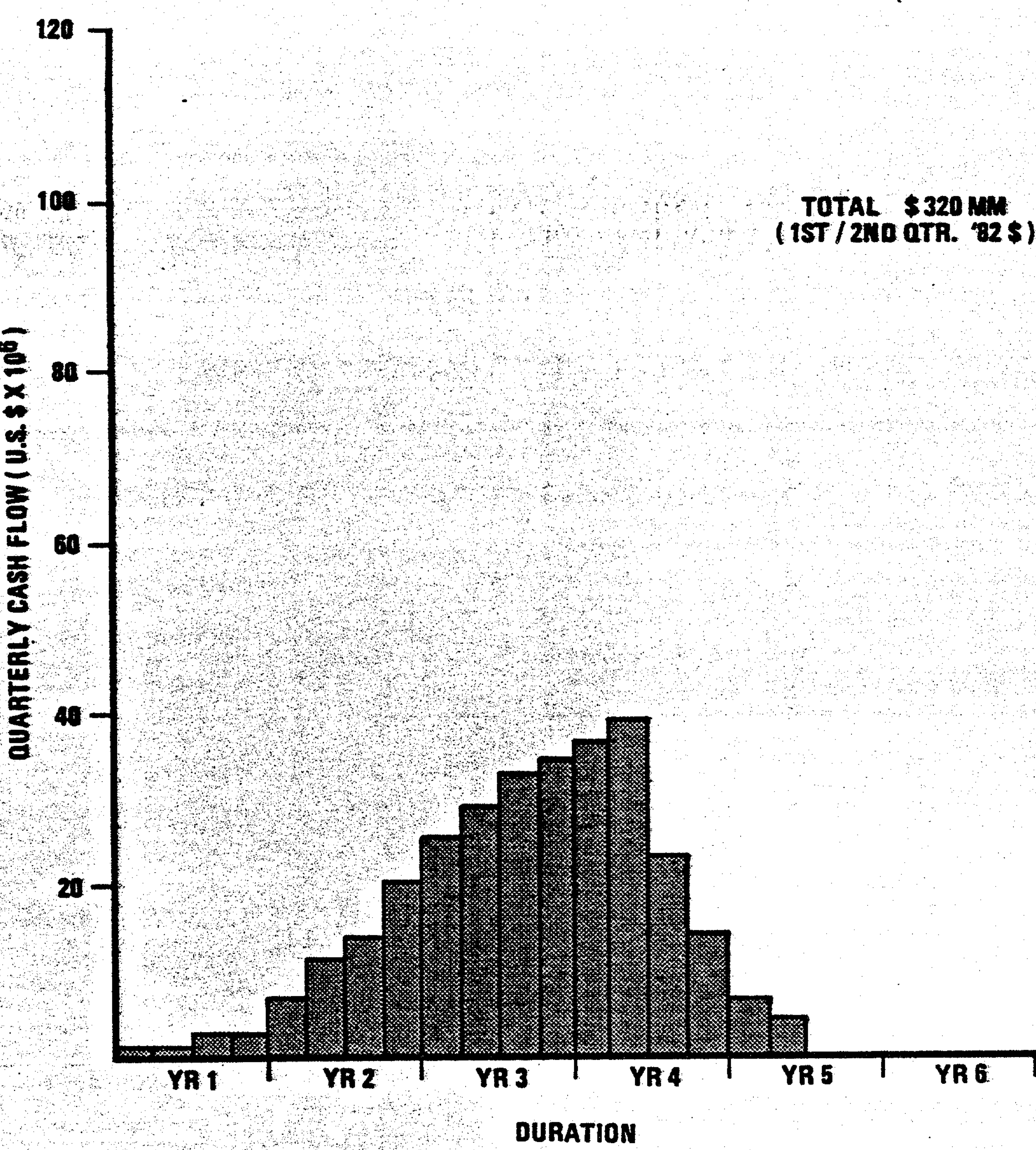


Figure 10-9

Southern Bering Sea Production Systems Study

CASH FLOW
ONSHORE FACILITIES
RESUPPLY BASE & AIRSTRIP

Job 15154 BECHTEL June 1982

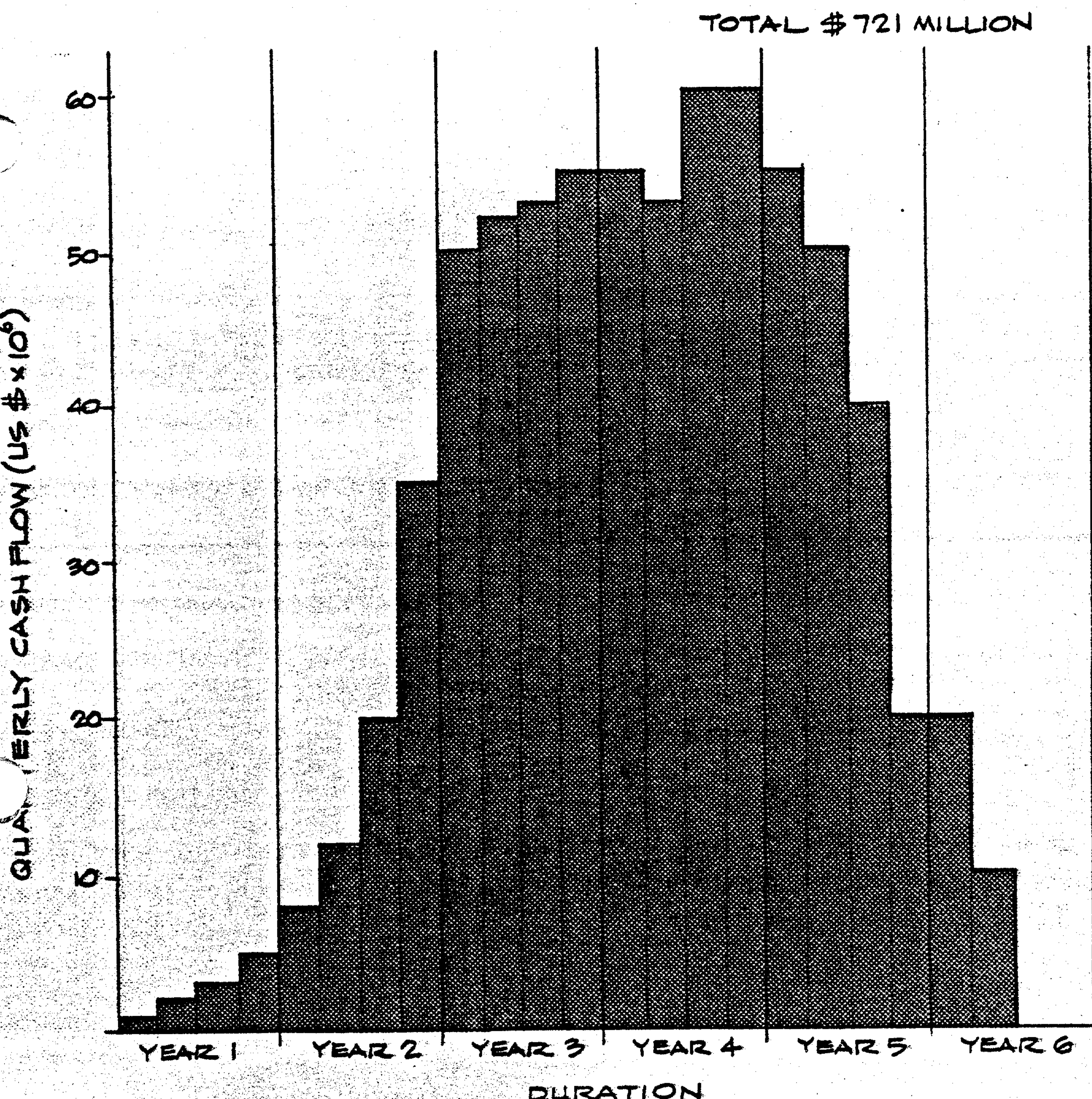


Figure 10-10

SOUTHERN BERING SEA PRODUCTION SYSTEM STUDY

ESTIMATED CASH FLOW
ST. GEORGE OR NAVARIN
STORAGE AND LOADING STRUCTURE
3 MILLION BARRELS

JUNE 1982

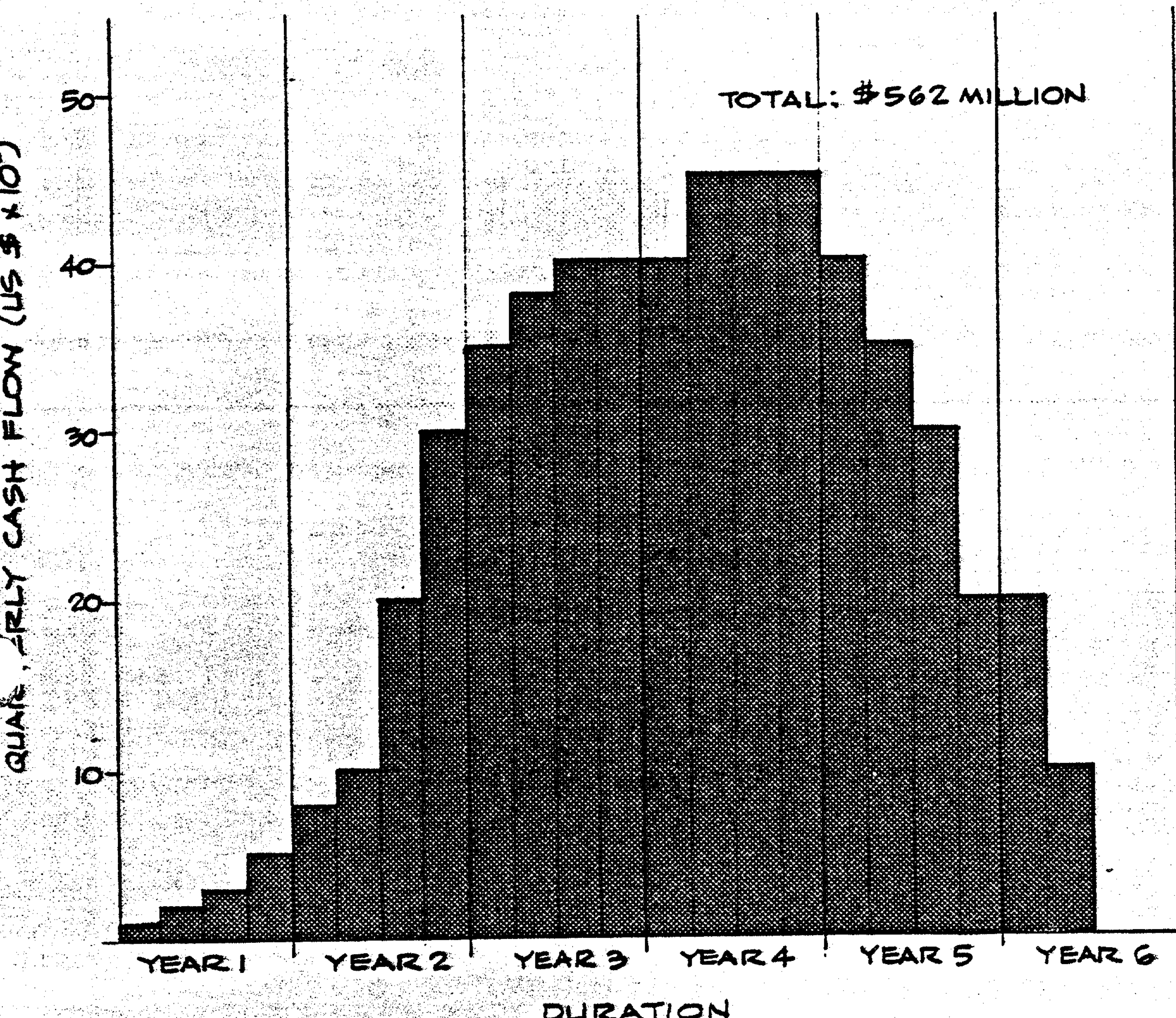


Figure 10-11

SOUTHERN BERING SEA PRODUCTION SYSTEM STUD

ESTIMATED CASH FLOW
HORTH ALEUTIAN
STORAGE AND LOADING STRUCTURE
2 MILLION BARRELS

JUNE 1982

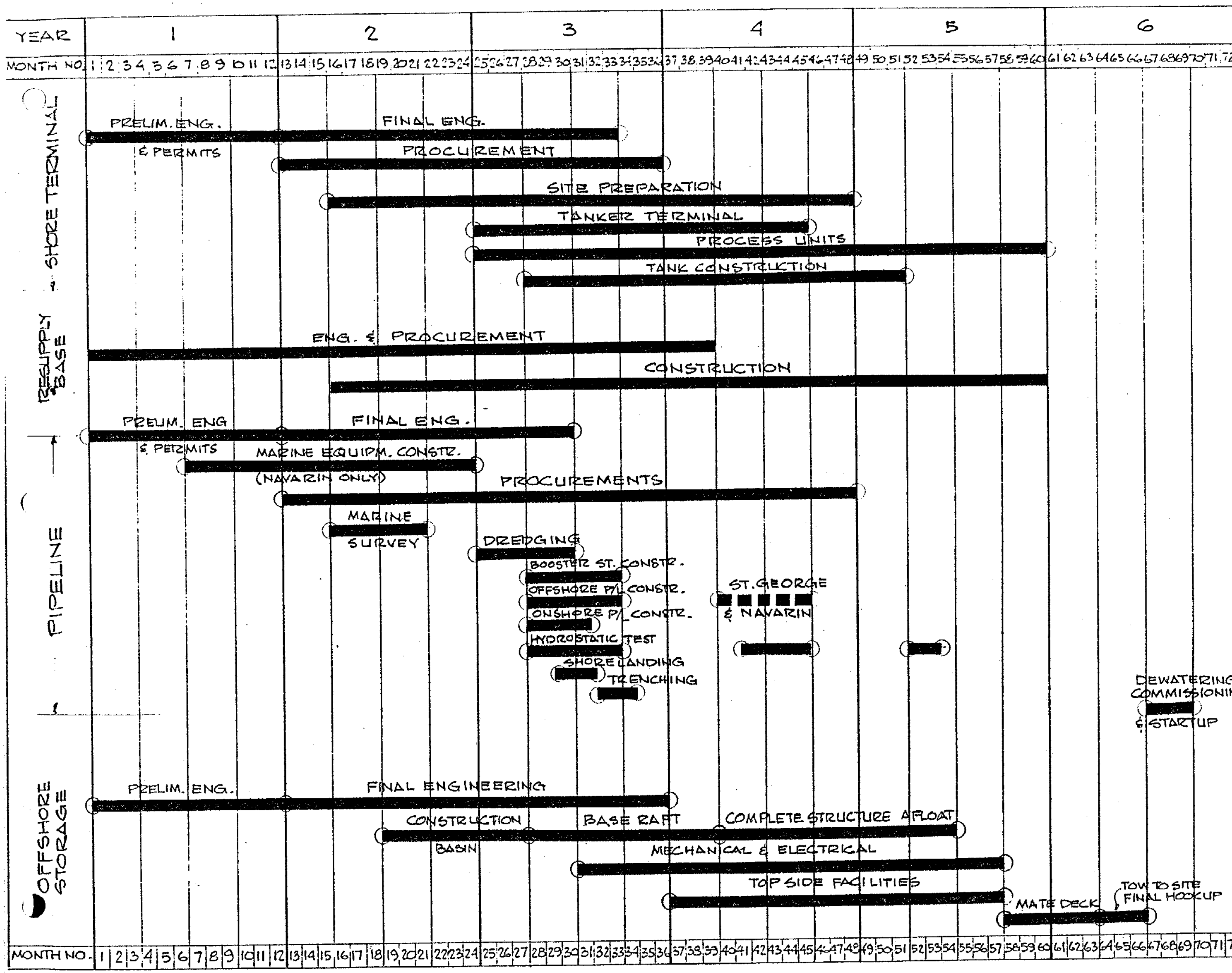


Figure 10-12

SOUTHERN BERING SEA PRODUCTION SYSTEMS STUDY

MASTER SCHEDULE TRANSPORTATION SYSTEM

Table 10 - 1

BOOSTER STATION COSTS

(U.S. Dollars x 10^6)

Item	Throughput 100 MBPD	300 MBPD
Topside Materials and Fabrication	30	45
Jacket 3,000 Tons at $5,500/ton	16.5	16.5
Piling 2,500 Tons at $1,800/ton	4.5	4.5
Transportation	2.0	2.0
Mob/Demob	3.0	3.0
Installation 30 days at $200,000/day	6.0	6.0
	62.0	77.0

Table 10 - 2
COST BREAKDOWN
North Aleutian P/L - 24" O.D.
(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		16
Laybarge		
o Mob/Demob	1 - 2nd Gen.	19
o Operation	1-1 Season @ $365,000/day	67
Barge Support Fleet		
o Mob/Demob		3
o Operation	1-1 Season @ $72,000/day	13
Work Barges		
o Mob/Demob	1 @ $12,000,000	12
o Operation	2 Tie-ins @ $3,000,000 ea	6
	1 Repair @ $4,000,000 ea	4
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tender		
o Mob/Demob		1
o Operation	1-1 Season @ $60,000/day	11
Marine Survey	(Allowance)	3
Total Mat. and Labor Offshore 26 nm)		263
Material - Herendeen Bay (23 nm)		14
Mat. and Labor - Onshore (16 sm)		30
Total Field Cost		307
Home Office @ 10%		31
Total Home Office and Field		338
Contingency @ 30%		101
Total Capital Cost		440

Table 10 - 3
COST BREAKDOWN
St. George P/L - 24" O.D.
(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		124
Laybarge		
o Mob/Demob	2-2nd Generation	37
o Operation	2-2 Seasons @ $365,000/day ea.	267
Barge Support Fleet		
o Mob/Demob		5
o Operation	2-2 Seasons @ $72,000/day ea.	53
Barge Winter Layover	2-1 Season @ $20,000/day ea.	8
Work Barges		
o Mob/Demob	2 @ $12,000,000 ea.	24
o Operation	8 Tie-ins @ $3,000,000 ea.	24
	2 Repairs @ $4,000,000 ea.	8
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tenders		
o Mob/Demob		2
o Operation	2-2 Seasons @ $60,000/day ea.	44
Marine Survey	(Allowance)	7
Total Mat. and Labor Offshore (203 nm)		711
Material - Herendeen Bay (23 nm)		14
Mat. and Labor Onshore (16 sm)		30
Total Field Cost		755
Home Office @ 10%		76
Total Home Office and Field		831
Contingency @ 30%		250
Total Capital Cost		1,080

Table 10 - 4
COST BREAKDOWN
Navarin P/L - 24" O.D.
(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		346
Laybarge		
o Mob/Demob	2-3rd Gen. (Barge Construction Cost)	320
o Operation	2-2 Season @ $391,000/day ea.	286
Barge Support Fleet		
o Mob/Demob		8
o Operation	2-2 Seasons @ $144,000/day	105
Barge Winter Layover	2-1 Season @ $100,000/day ea.	37
Work Barges		
o Mob/Demob	3 @ $12,000,000 ea.	36
o Operation	6 Tie-ins @ $3,000,000 ea.	18
	4 Repairs @ $4,000,000 ea.	16
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tenders		
o Mob/Demob		2
o Operation	2-2 Seasons @ $60,000/day ea.	44
Marine Survey	(Allowance)	10
Booster Station		62
Total Mat. and Labor Offshore (572 nm)		1,398
Material - Herendeen Bay (23 nm)		14
Mat. and Labor Onshore (16 sm)		30
Total Field Cost		1,440
Home Office @ 10%		144
Total Home Office and Field		1,586
Contingency @ 30%		476
Total Capital Cost		2,100

Table 10 - 5
COST BREAKDOWN
North Aleutian P/L - 36" O.D.
(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		37
Laybarge		
o Mob/Demob	1-2nd Gen.	19
o Operation	1-1 Season @ $365,000/day	67
Barge Support Fleet		
o Mob/Demob		3
o Operation	1-1 Season @ $72,000/day	13
Work Barges		
o Mob/Demob	1 @ $12,000,000	12
o Operation	2 Tie-ins @ $3,000,000 ea.	6
	1 Repair @ $4,000,000	4
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tender		
o Mob/Demob		1
o Operation	1-1 Season @ $60,000/day	11
Marine Survey	(Allowance)	3
Total Mat. and Labor Offshore (26 nm)		284
Material - Herendeen Bay (23 nm)		33
Mat. and Labor Onshore (16 sm)		46
Total Field Cost		363
Home Office @ 10%		36
Total Home Office and Field		399
Contingency @ 30%		120
Total Capital Cost		520

Table 10 - 6

COST BREAKDOWN

St. George P/L - 36" O.D.

(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		252
Laybarge		
o Mob/Demob		68
o Operation	2nd Gen., 2-2 2/2 Seasons and 1-1 Season @ $365,000/day	334
Barge Support Fleet		
o Mob/Demob		8
o Operation	2-2 Seasons and 1-1 Season @ $72,000/day ea.	66
Winter Layover	2-1 Season @ $20,000/day ea.	8
Work Barges		
o Mob/Demob	3 @ $12,000,000 ea.	24
o Operation	10 Tie-ins @ $3,000,000 ea.	30
	2 Repairs @ $4,000,000 ea.	8
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tenders		
o Mob/Demob		2
o Operation	2-2 Seasons @ $60,000/day ea.	44
Marine Survey	(Allowance)	7
Total Mat. and Labor Offshore (203 nm)		959
Material - Herendeen Bay (23 nm)		29
Mat. and Labor Onshore (16 sm)		46
Total Field Cost		1,034
Home Office @ 10%		103
Total Home Office and Field		1,137
Contingency @ 30%		341
Total Capital Cost		1,480

Table 10 - 7
COST BREAKDOWN
Navarin P/L - 36" O.D.
(U.S. Dollars x 10^6)

Item	Basis	Cost
P/L Material (Incl. Freight)		710
Laybarge		
o Mob/Demob	1-2nd Generation	19
o Mob/Demob	2-3rd Gen. (Barge Construction Cost)	320
o Operation	1-2nd Gen. for 1 Season @ $365,000/day	67
	2-3rd Gen. for 2 Seasons @ $391,000/day ea.	286
Barge Support Fleet		
o Mob/Demob		11
o Operation	1-2nd Gen for 1 Season @ $72,000/day 2-3rd Gen for 2 Seasons @ $144,000/day ea.	126*
Barge Winter Layover	2-1 Season @ $100,000/day ea.	37
Work Barges		
o Mob/Demob	3 @ $12,000,000 ea.	36
o Operation	8 Tie-ins @ $3,000,000 ea.	24
	4 Repair @ $4,000,000 ea.	16
Pipe Trenching	(Allowance)	46
Shore Approaches	(Allowance)	60
Dredging	(Allowance)	2
Subs and Tenders		
o Mob and Demob		2
o Operation	2-2 Seasons @ $60,000/day ea.	44
Marine Survey	(Allowance)	10
Booster Station		77
Total Mat. and Labor Offshore (572 NM)		1,893
Material - Herendeen Bay (23 NM)		29
Mat. and Labor Onshore (16 SM)		46
Total Field Cost		1,968
Home Office @ 10%		197
Total Home Office and Field		2,165
Contingency @ 30%		650
Total Capital Cost		2,800

*Includes additional $7 MM for extra pipe haul vessels

Table 10 - 8

CAPITAL COST ESTIMATES - PIPELINES

(U.S. Dollars x 10^6)

FACILITIES	Throughputs 100 MBPD	300 MBPD
1. PIPELINE TO NAVARIN BASIN		
PIPELINE MATERIAL	530	1,080
CONSTRUCTION	1,570*	1,720**
NAVARIN TOTAL	2,100	2,800
2. PIPELINE TO ST. GEORGE BASIN		
PIPELINE MATERIAL	200	420
CONSTRUCTION	880	1,060
ST. GEORGE TOTAL	1,080	1,480
3. PIPELINE TO N. ALEUTIAN BASIN		
PIPELINE MATERIAL	57	120
CONSTRUCTION	383	400
N. ALEUTIAN TOTAL	440	520

* INCLUDES COST OF BOOSTER STATION IN AMOUNT OF \$62 x 10^6

** INCLUDES COST OF BOOSTER STATION IN AMOUNT OF \$77 x 10^6

NOTE: All costs include home office at 10% and contingency at 30%.

Table 10 - 9
CAPITAL COST ESTIMATES
ONSHORE FACILITIES

FACILITIES	ESTIMATED COST (U.S. Dollars X 10^6) 100 MBPD	300 MBPD
1. CRUDE OIL STORAGE AND TANKER LOADING FACILITIES*		
TOPPING PLANT	23	23
PROCESS UNITS	81	142
STORAGE TANKS	59	152
MARINE FACILITIES (TRESTLE, BERTH AND TUGS)	118	153
POWER AND DISTRIB. SYSTEMS	140	163
BALLAST WATER TREATMENT	6	12
SITE PREP. AND OFFPLOT	69	71
BUILDINGS AND PERMANENT HOUSING	54	54
COMMUNICATIONS	10	10
TOTAL TERMINAL	560	780

NOTE: Total terminal cost for a 10^6 BPD facility is estimated to be $1,800 x 10^6 including 7% home office costs and 35% contingency.

	ESTIMATED COST*** (U.S. Dollars X 10^6)
2. RESUPPLY BASE AND AIRSTRIP**	
CARGO QUAY AND FUEL DOCK	63
STORAGE TANKS	15
BUILDINGS AND PERMANENT HOUSING	73
EMERGENCY POWER AND DISTRIBUTION	58
SITE PREPARATION ROADS AND OFFPLOT	98
AIRSTRIP	7
PERMANENT MOBILE PLANT EQUIPMENT	6
TOTAL RESUPPLY BASE AND AIRSTRIP	320

*SITE: Lefthand Bay
**SITE: Herendeen Bay
***Applies to all terminal throughputs

Except as noted, all costs include 10% home office costs and 30% contingency.