STATE-OF-THE-ART REPORT

ON

GUYED TOWER PLATFORMS

PREPARED FOR

NATIONAL BUREAU OF STANDARDS

U. S. DEPARTMENT OF COMMERCE

Brown & Root Development, Inc.

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1. GENERAL CONCEPTS

1.1 INTRODUCTION

The guyed tower has been developed as a deepwater drilling and production platform (Figure 1-1). For water depths beyond about 1,000 ft., it is expected that the guyed tower can provide an economic advantage over conventional fixed platforms. Since the guyed tower permits conventional drilling and production operations, it also can offer advantages over floating system that use subsea wellheads. Drilling operations are conducted from the deck of the tower and conventional field processing equipment and utilities are in modules on a two or three level deck.

Several engineering studies have established the conceptual feasibility of the guyed tower (1-5). Scale model tests both in the ocean and in wave tanks have verified the theoretical aspects of guyed tower behavior. The first commercial application of on offshore guyed tower for drilling and production of hydrocarbon reserves is being made by Exxon Co. The structure is to be installed during the summer of 1983 at the Lena prospect (Mississippi Canyon Block 280) in 1,000 ft. of water in the Gulf of Mexico (Fig. 1-2).

The guyed tower belongs to a class of structures commonly referred to as Compliant. The basic idea of compliancy is that the platform is permitted to move in response to applied environmental forces rather than rigidly resist them as in a conventional fixed platform.

1.2 DYNAMIC COMPLIANCY

It is accepted practice to design shallow water platforms using static methods of analysis. The static approach is adequate since the fundamental natural period of such platforms is much smaller than the predominant periods of the waves at which significant energies are contained. However, as the ratio of the natural period of the platform to the period associated with the significant energy in the design sea state increases, inertial forces become important. These concepts are further illustrated with reference to specific examples.

The action of lateral wave forces due to the design storm on a shallow water fixed platform is known to be static. The distribution of wave forces and associated inertial loads on a deepwater fixed platform is shown in Figure 1-3(a). Finally both the wave forces and inertial loads on a guyed tower are shown in Figure 1-3(b). The following important observations can be made. The inertial forces acting on the deepwater fixed platform are quite significant. Furthermore, the inertial forces act in the same direction as the wave forces and the total force for which the platform must be designed is increased. The guyed tower exhibits an interesting phenomenon in which the inertial forces act in a sense opposite to the wave forces and hence decrease the magnitude of the lateral forces for which the platform must be designed. The first example is typical of most structures but the behavior of the guyed tower is quite different from that usually encountered in engineering practice. While in most instances dynamic response leads to amplified design forces, in the case of guyed tower dynamic action is utilized to reduce the design forces. The behavior of the guyed tower system is commonly referred to as compliancy. An alternate designation would be dynamic deamplification.

The above concepts are graphically illustrated in Figure 1-4. The ordinate in this figure represents the ratio of the dynamic lateral wave force to the force computed assuming the platform is rigid. The abscissa represents the ratio of the fundamental natural period of the platform to the period of the exciting forces assuming the latter to be periodic. For purposes of discussion this amplification diagram is divided into four regions. In region I, the amplification of the wave forces is negligible. Shallow water platforms fall into this category. The period of the platform must be less than about twenty percent of the design wave period so that the wave forces can be assumed to act in a static manner. Region II is characterized by dynamic amplification. Deepwater fixed platforms fall under this region. The upper limit of this region is governed by a number of factors including fatigue, practical design and construction considerations and above all platform cost. At the present state-of-the-art it is believed that the platform period can be as high as forty to fifty percent of the period of the design wave. Region III is characterized by high dynamic amplifications. Economic considerations discourage design and construction of such structures. Compliant structures such as a guyed tower, buoyant tower or tension leg platform belong to region IV. Note in particular that the design forces for structures in this region are only a fraction of the forces computed assuming static behavior of the structure.

1.3 GUYED TOWER DESCRIPTION

The guyed tower platform consists of four major components: deck, tower, foundation and mooring system (Figure 1-5). The deck can be of the conventional modular type and can be installed using conventional equipment and procedures. The tower supports the deck, protects the conductors and risers, and serves as a template for driving piles if a pile foundation is employed. The tower is similar to a fixed platform jacket but has generally a uniform cross section. It is designed using the same principles and procedures used in the design of deepwater fixed jackets. Large permanent buoyancy tanks are sometimes built into the upper portions of the tower to reduce vertical loads on the foundation system. The permanent buoyancy also provides additional restoring forces.

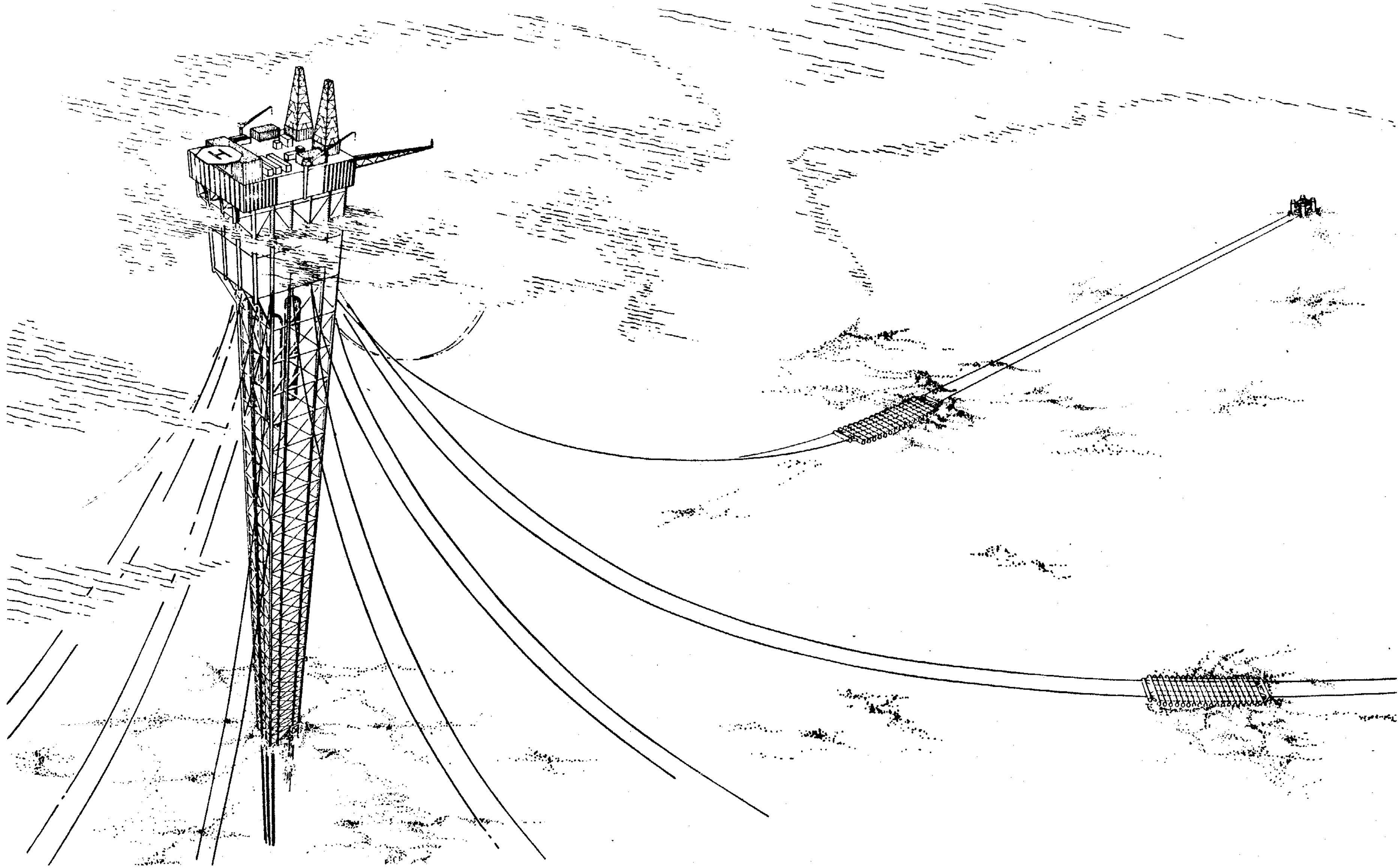
Two types of foundation systems have been proposed. Earlier work on guyed tower platform utilized a vertical bearing foundation called a spud can (Figure 1-6). It is basically a truss reinforced stiffened shell which is artificially forced into the ocean bottom immediately after installation until the required load carrying capability is attained. A procedure has been devised to force the can downward by adding an overload in the form of a heavy drilling mud into the spud can. Once the deck load has been placed and the tower has reached the desired depth of penetration, the drilling mud is pumped out of the spud can. Extensive analytical studies and model test have been conducted to predict the penetration of the spud can into the soil. The spud can foundation system has been used in the test guyed tower program conducted in the Gulf of Mexico by Exxon Production Research Company.

More recent work identified the pile foundation as a viable alternate to the spud can (Fig. 1-7). Experience with the performance and installation aspects of piles in conventional platforms make the pile foundation particularly attractive.

The piled foundation consists of sufficient number of ungrouted piles usually located near the center of the tower. The piles are spaced as close as practically possible to reduce foundation fixity at the base. The piles are attached to the tower at its top by welded connections. The pile foundation is described further in Section 2.5.1.

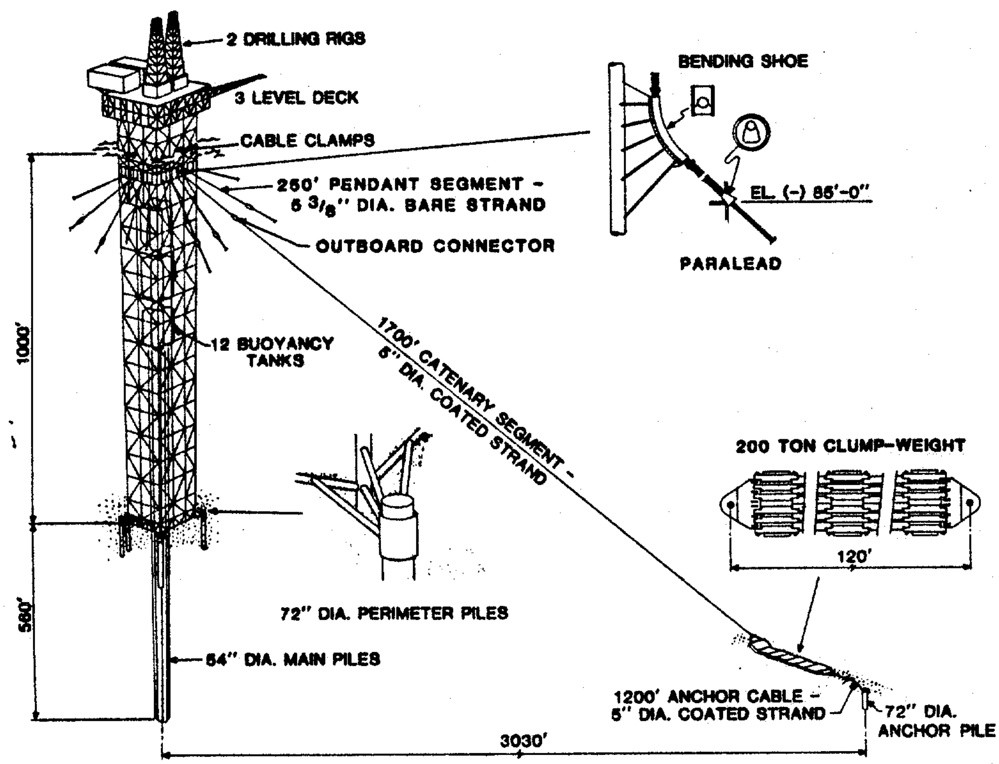
The mooring system consisting of several guylines attached to the tower near mean water level and arranged radially provide lateral support for the tower. The guylines extend from the tower to clumpweights on the ocean floor. Anchor lines will connect the clumpweights to anchoring devices installed on the ocean floor. Under normal operating conditions the clumpweights will remain on the ocean floor, and the platform motions will be nearly imperceptible. During a severe storm, the weights on the stormward side of the tower will lift off bottom and will soften the guying sytems. This will permit the tower and the guying systems to absorb the energy associated with the large wave loads without appreciable increase on the guyline tension.

The mooring system described above which incorporates clumpweights is the one widely considered for guyed tower design. However, it is possible to design a mooring system without clumpweights under certain circumstances, especially for applicatons in the deeper waters. The merits and demerits of such a mooring system are discussed in a later section.



GUYED TOWER PLATFORM

FIGURE 1-1



EXXON LENA GUYED TOWER
(REF. 8)

FIGURE 1-2

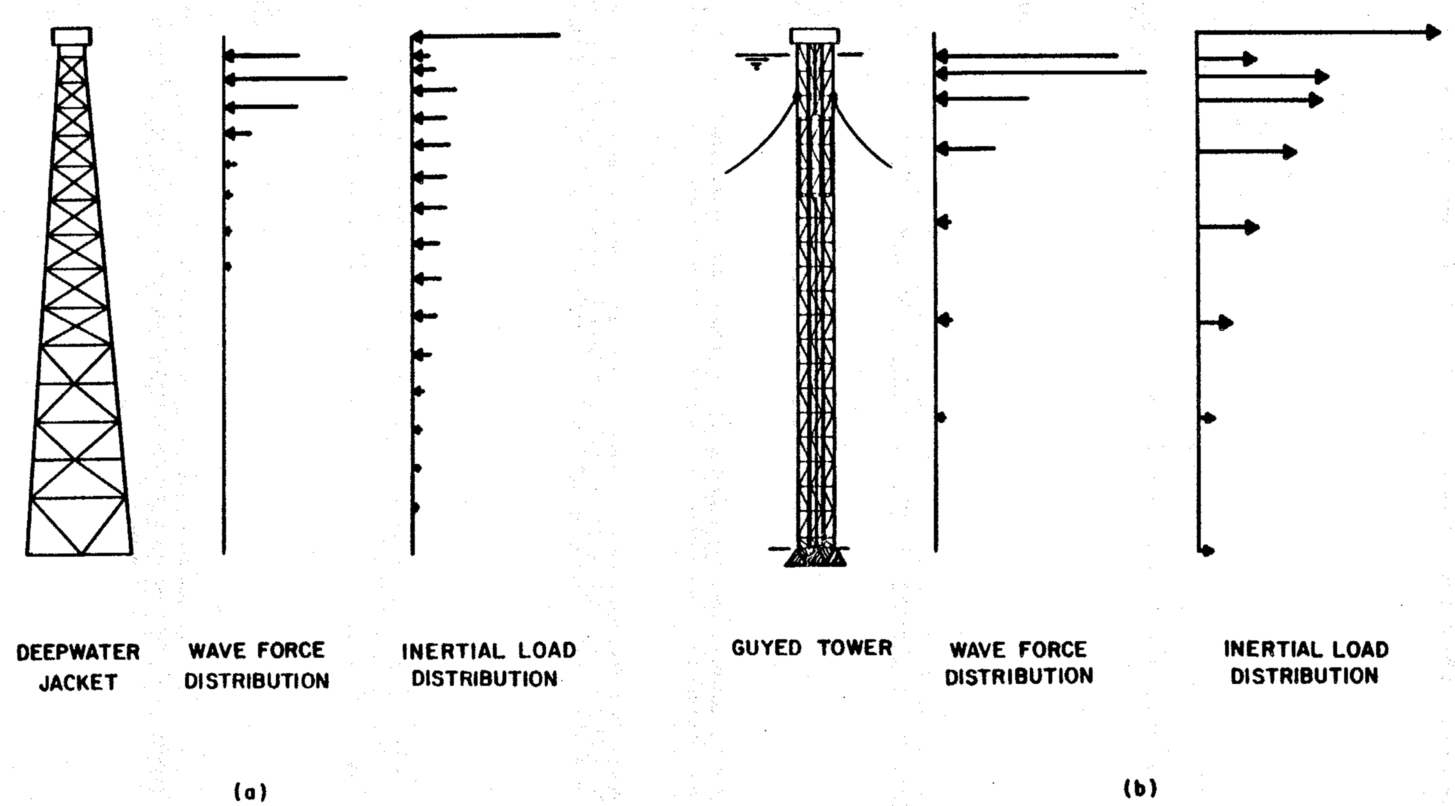


FIGURE 1-3

WAVE AND INERTIAL FORCES ON JACKETS AND GUYED TOWERS

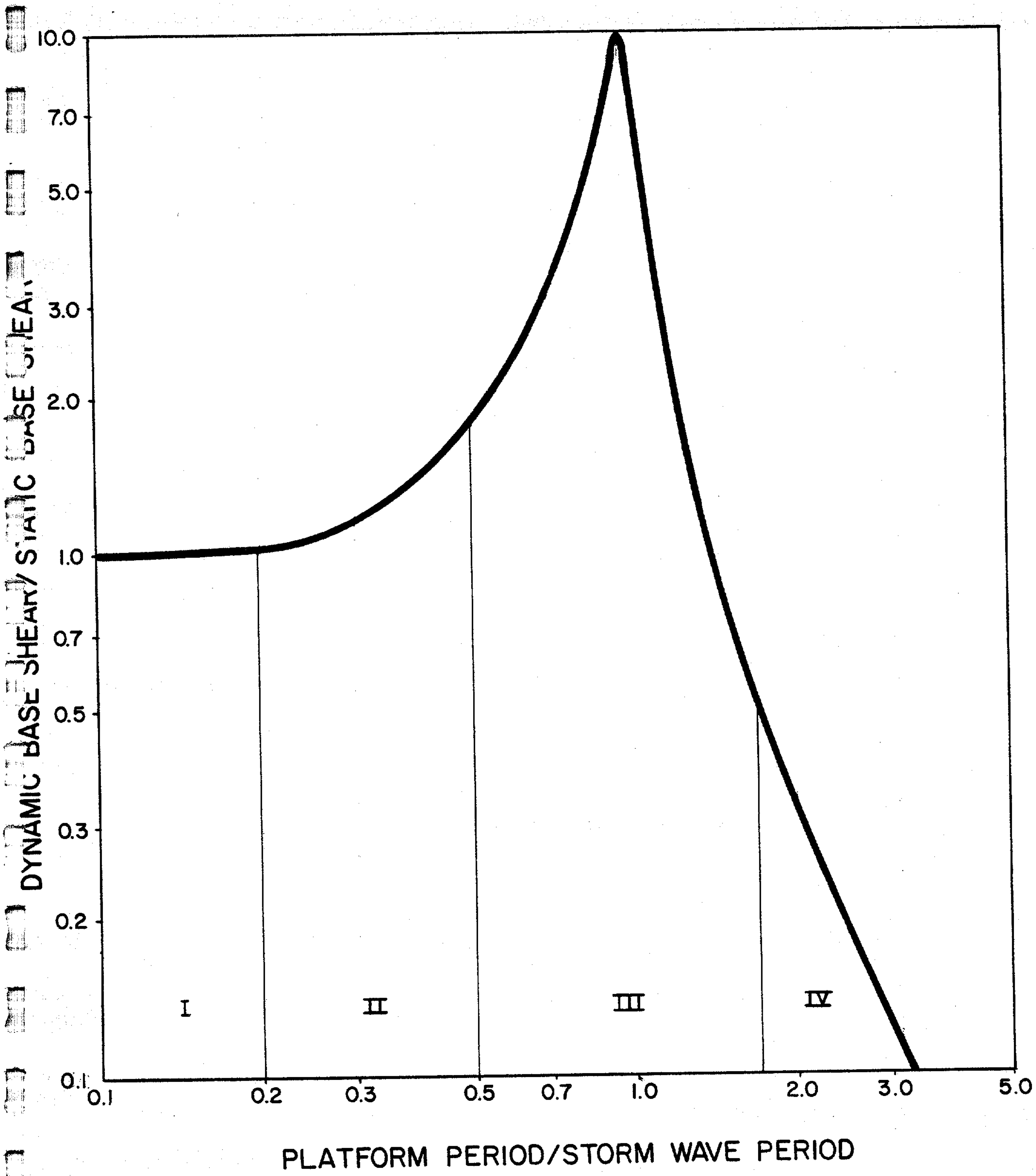
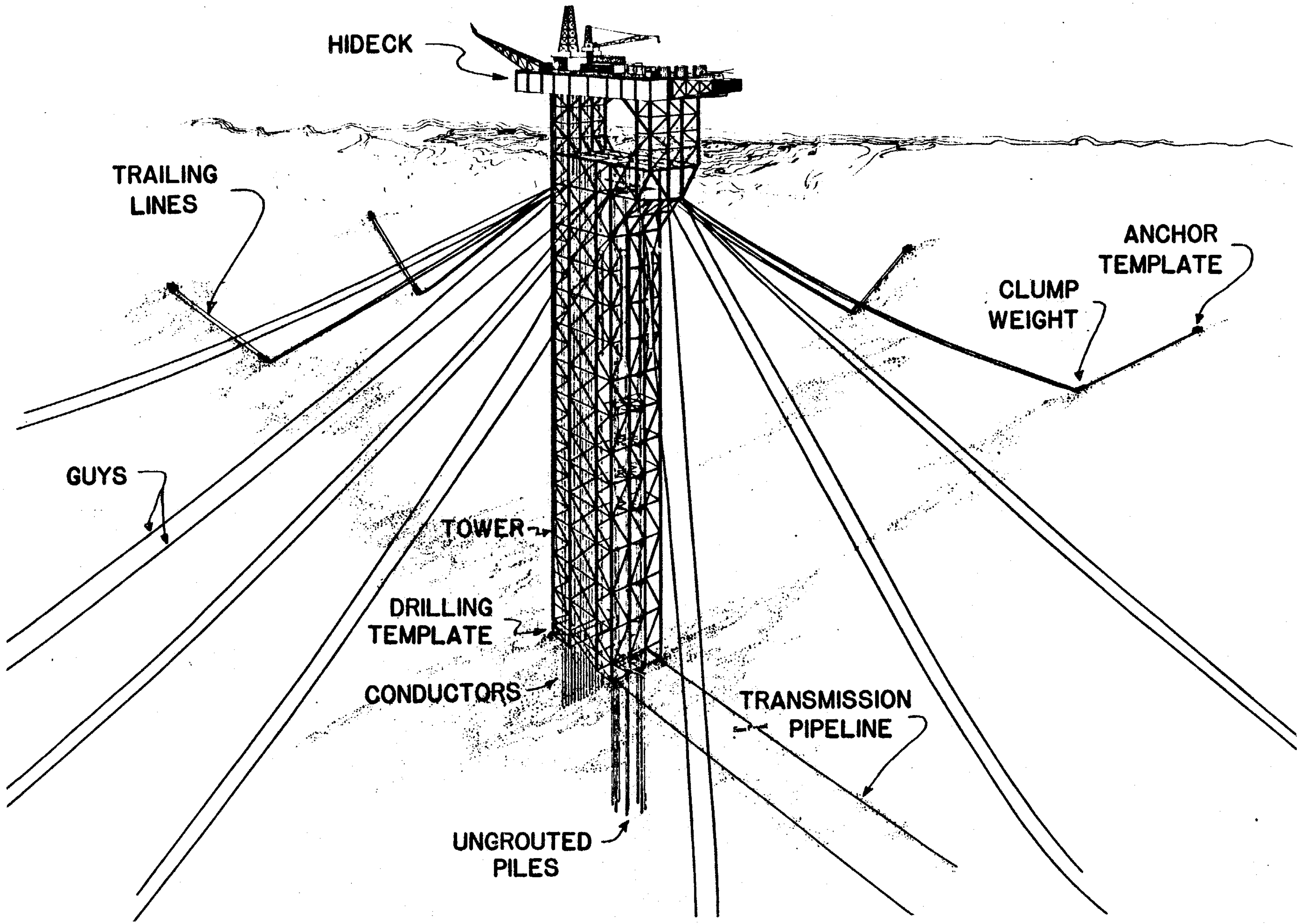


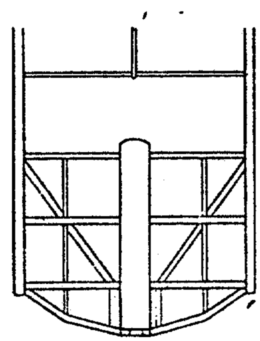
FIGURE 1-4

WAVE RESPONSE SPECTRUM



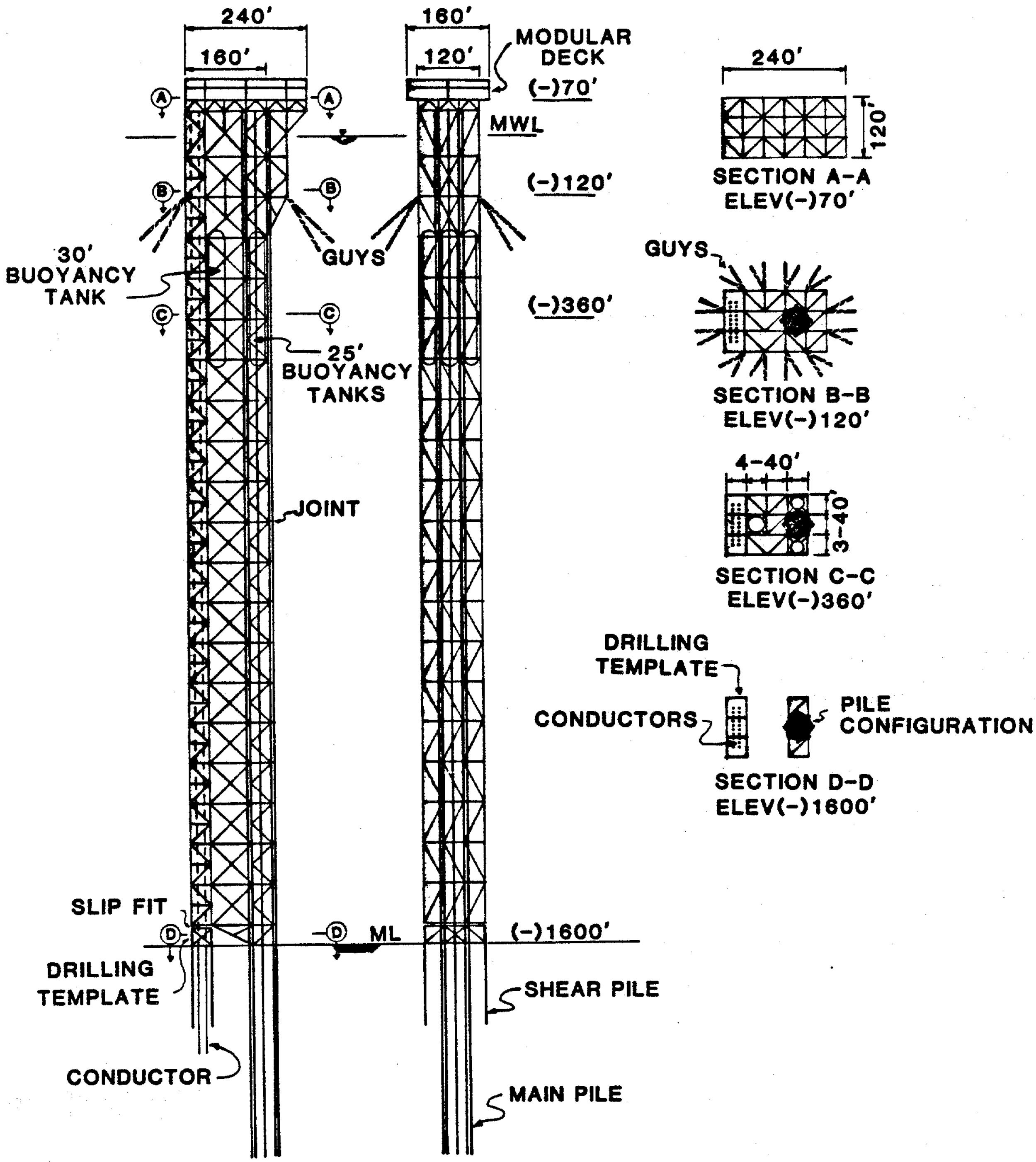
GUYED TOWER COMPONENTS

FIGURE 1-5



SPUD CAN

FIGURE 1-6



PILE FOUNDED GUYED TOWER

WITH DRILLING TEMPLATE AND MODULAR DECK

FIGURE 1-7

2. DESIGN CONSIDERATIONS

This section of the report discusses the major design considerations with emphasis on those factors which are particularly important for guyed towers.

2.1 LOADS

The following types of loads should be considered in the design.

1. Dead Loads - These are the weights of the platform structure, permanent equipment and appurtenant structures. The platform weight inclues platform weight in air with appropriate provision for piles, grout, flooded water, etc.

2. Buoyancy and hydrostatic forces.

3. Live Loads - These are loads imposed on the platform during its use and which may change. Examples of such loads are drilling and production equipments which could be added or removed from the platform, weight of supplies and storage items, and forces generated from operations such as drilling, material handling, etc.

4. Environmental Loads - These are loads imposed on the platform by natural phenomena such as wind, wave, current, earthquake, soil movement, ice, etc. The dynamic effects associated with the response of the guyed tower to these loads should also be considered.

5. Construction Loads - These loads result from fabrication, loadout, transportation and installation of the platform and its subsystems.

2.2 LOADING CONDITIONS

Several representative loading conditions should be investigated during design. Both operational and extreme design sea states should be investigated and in each case several directions of wave approach should be considered. Both maximum and minimum conditions of deck payload and critical positions of drilling rig positions should also be investigated.

Some of the important loading conditions are identified below.

(1) Operating Condition,
(2) Design Condition,
(3) Damaged Condition, and
(4) Extreme Condition.

A more detailed treatment of these loading conditions and their significance on guyed tower design are presented in a later section.

2.3 DECK

The deck configuration is primarily determined by the functional requirements related to the drilling and production operations. It should also be compatible with the tower configuration, which is square or approximately square in most cases. In locating the elevation of the lowest deck consideration should be given to the setdown due to the substantial lateral deflection of the tower under extreme storm.

The structural design of the deck can be done following procedures similar to that used for fixed platforms. The dynamic effects of wind should be investigated and accounted for in an appropriate manner. The inertia loads on the deck produced by the motions of the platform should also be considered in the design of the deck.

In order to reduce the second order forces, the design may provide for the uncoupling of the tower and well conductors. In such cases, the design should provide space for the vertical movement of the wellheads.

2.4 TOWER

The overall design of the tower should follow procedures used in the design of jackets (6). The cross sectional dimensions of the tower are governed by the following considerations: (1) the tower should be large enough to support the deck; (2) the tower should provide sufficient space for the conductors and the foundation system consisting of a number of piles; and (3) the tower should be sufficiently stiff so that its flexural period is less than about six seconds to avoid amplification of wave loads in that mode and hence minimize member forces.

The cross section of the guyed tower can be kept uniform since an increased base is not required. The bracing patterns and member sizes are selected using fixed platform design practice. The member sizes in the upper part of the tower are governed by gravity loads, local wave loads and the pretension in the mooring system. In the bottom part of the tower the member sizes are governed by hydrostatic loadings. In the middle portion of the tower, the flexural mode of the tower produces most of the stresses. Fabrication and installation considerations will govern the design of members locally.

2.5 FOUNDATION

As mentioned in Section 1, two types of foundation systems have been proposed for the guyed tower, the spud can and piled foundation. The spud can is basically a truss-reinforced stiffened shell (Figure 1-6) which is artificially forced into the ocean bottom immediately after installation until the desired load carrying capability is attained. The can is forced downward by adding an overload in the form of a heavy drilling mud to the spud can. Once the deck load has been placed and the tower has reached the desired depth of penetration, the drilling mud is pumped out of the spud can. Both analytical and experimental studies have been performed to predict the load-settlement behavior of the spud can. The major disadvantages of this foundation system are that the installation of the spudcan is difficult and that the long term vertical settlement of the foundation system is of concern.

More recent work identified the pile foundation as a viable alternate to the spud can system. Experience with the performance and installation of piles in conventional platforms make such a foundation the favored concept.

2.5.1 Pile Founded Guyed Tower

The basic idea of the guyed tower is that it acts as a rigid tower pinned at the base in its fundamental sway mode. A pile foundation system can be designed to act as a pivot which permits rotation of the tower. In the design of conventional deepwater fixed platforms the increased stiffness and foundation capacity are obtained by increasing the size of the jacket and by spacing the piles as far apart as possible. In the case of the guyed tower an opposite approach is used to decrease the stiffness and thereby increase the natural period. Specifically, the tower is designed to be slender, and the piles are closely spaced and ungrouted. This design strategy virtually uncouples the lateral stiffness of the tower and the bending stiffness of the piles. The tower can be thought of as being suspended from the piles since the pile terminations where the load transfer is effected are located above the mean water level. The axial flexibility of the piles in combination with their close spacing leads to the required pivotal action.

Figure 2-1(a) shows a two dimensional representation of the guyed tower. Figure 2-1(b) shows a physical model which simulates the action of the piles. The piles are replaced with springs having the same stiffness as the axial stiffness of the piles. Finally Figure 2-1(c) shows the analytical model in which the tower is represented as a rigid bar with masses lumped at various locations. In this model both the stiffness provided by the piles and the rotational stiffness provided by the foundation are represented by a single rotational spring. The above analytical model is designed to illustrate how the compliant behavior is achieved in a pile founded guyed tower.

The foundation system for the guyed tower consists of a number of deep penetrating vertical piles. The number and size of piles is primarily dictated by the total foundation load to be carried, pile capacity and installation considerations. Group efficiency of piles must also be considered in selecting the foundation configuration. The axial loads on the piles are due to three components: (1) the net effect of structural dead weight plus the deck payload less the buoyancy; (2) the vertical component of the forces in the mooring system; and (3) the axial forces due to environmental loadings. The first two items produce about sixty to eighty percent of the axial load on the most heavily loaded pile. The vertical load due to pretension in the mooring system is quite significant.

The gravity loads and the contribution of the pretension produce constant axial loads on the piles. Environmental loads produce oscillatory axial forces. Since the oscillatory axial loads are less than the constant compressive loads, the piles in a guyed tower are always under compression. This is in contrast to fixed platforms in which the piles are also subjected to tension under design environmental conditions.

Since most of the environmental loads are resisted by the mooring system, the shear and moment at pile head are not significant. Hence an optimum design of the foundation system should reduce the axial load on the piles. A good design strategy is to reduce the static component of vertical load on the foundation by use of additional buoyancy tanks in the tower. Such buoyancy tanks must be located sufficiently below the mean water level so that the wave forces can be reduced.

The design of the pile foundation can be based on conventional design procedures, except that the dynamic behavior of the soil-pile system must be considered. In particular the cyclic degrading behavior of the soil under lateral loads should be considered. Besides the maximum stress criteria, the fatigue of the pile should also be investigated.

Torsional rigidity is an important consideration in the design of the foundation system. Since the torsional rigidity of the guyed tower is provided by the foundation system, additional rigidity can be provided by using shear piles. Conductors will also contribute to the shear and torsion capacity and hence should be included in the analytical model.

2.6 MOORING SYSTEMS

There are primarily two types of mooring systems. The first is commonly referred to as a catenary mooring system. In a catenary mooring, the mooring line exits the structure or vessel at a fairlead, hangs in the shape of a catenary until at some distance out from the fairlead it contacts the sea floor and is tangent to the sea floor at this point. The line then runs along the sea floor to an anchor which fixes the end of the mooring line. This type system is characterized by the fact that increases in line tension caused by increased forces at the fairlead are resisted by picking up additional line from the sea floor, and thus its behavior is governed by their unit weight.

The distributed clumpweight mooring (Fig. 2-2) can be viewed as an optimized conventional catenary mooring. The stiffness of a conventional mooring is determined by the sag of the suspended section which is always at a maximum due to the lifting action. The most effective way to increase the stiffness is to reduce the sag of the suspended section by reducing the unit weight of that section. This is accomplished by using wire rope, which is very light when compared to the weight per unit length of the clumpweight, pretensioned to about 30 percent of its breaking strength to reduce the sag to an acceptable amount.

2.7 DESIGN OF CLUMPWEIGHT MOORING SYSTEM

The mooring system is a distinguishing feature of the guyed tower and is the primary contributor to the lateral stiffness of the platform. Besides providing the required lateral stiffness, the mooring system with clumpweights also acts as a relief valve under extreme loads. The load-deflection relationship for a typical mooring system for lateral response is shown in Figure 2-3. It may be noted that the load-deflection behavior is essentially linear up to a certain level (Point A) of tower deflection which covers a wide range of operating environmental conditions. However, during the design storm the deflections of the tower are such that the tower responds in the nonlinear softening part of the load-deflection curve. The implications of the mooring system behavior on the tower response can be interpreted as follows. The decrease in the mooring system stiffness leads to an apparent increase in the instantaneous period of the platform and this places the tower response farther to the right in region IV of the response spectrum shown in Figure 1-4. This further reduces the dynamic amplification of environmental loads. Stated differently, unlike the case of a linear structure, an increase in environmental loads does not produce a corresponding increase in the tower response, especially in the resultant cable forces which support the tower. Thus the mooring system can be thought of as a relief valve which provides protection against overloads. The static behavior of the mooring system will be examined in more detail in a later section.

Analysis of guyed towers reported to date model the mooring system as a massless spring having nonlinear force displacement behavior derived from a static analysis of the mooring system. Such an idealization which uncouples the dynamic behavior of the tower and the mooring system is adequate for predicting overall response of the guyed tower and has been verified through the at sea model test reported in Reference 2. Analytical investigations currently in progress at Rice University also confirm this conclusion. However, the dynamic behavior of the individual guylines should be explicitly considered to predict the variation of tensions accurately (2, 7, 8).

The selection of a mooring system is of primary importance in a guyed tower design since the lateral resistance to the environmental loadings is almost totally provided by the mooring lines. In designs of guyed towers to date, a starting point for the mooring system design has been to obtain an estimate of the required initial lateral stiffness of the mooring system which will guarantee satisfactory tower response under operational sea states. The magnitude of this stiffness, referred to as the initial stiffness, is governed by the following factors: (1) the sway period of the tower should be approximately twice the period of the design wave; (2) the stiffness should be such that the tower deflections are acceptable under static environmental loads, namely wind and current loads; and (3) the mooring system should be sufficiently redundant such that the platform behavior is acceptable even with a specified number of mooring lines out of service when subjected to the design storm.

In addition to the required initial lateral stiffness an estimate of the required strength level of the mooring system is useful in selecting a preliminary mooring configuration. The term, strength level, is used to refer to the total lateral resistance that the mooring system must provide under extreme loading conditions before any significant reduction in the mooring array stiffness occurs. Factors influencing the required strength level are: (1) the maximum wind load occurring during the extreme storm condition; (2) the nonzero mean of the combined current and wave loadings; and (3) the P-Delta effects associated with the weight of the deck and tower structure.

Mooring systems for guyed towers may consist of sixteen to twenty-four mooring lines. Two systems which utilize twenty-four mooring lines are shown in Figures 2-4 and 2-5. The system shown in Figure 2-4 is arranged in twenty-four evenly spaced radial directions. In Figure 2-5 the mooring lines are paired in twelve radial directions. The choice of the number of mooring lines in the system depends on many practical considerations which, in addition to the redundancy requirement mentioned previously, include items such as the limitations and availability of installation equipment, the cost of installation and materials, the material availability, and the long term operation considerations. The choice between single and paired lines is governed by such factors as the topography in the area of the platform, the time required for the installation of the mooring lines, the lifting capacities of the installation equipment, and the clear area requirements surrounding the platform.

Figure 2-6 shows the arrangement of a single mooring line. It consists of an anchoring system, anchor line, clumpweight, and guyline. As will be discussed later, a distributed clumpweight offers many advantages. If a system of single lines is utilized, a single anchor pile per line may be used (1, 3). The anchor line termination is located below the mudline near the midpoint of the pile. Locating the termination below the mudline provides a very efficient means of lateral load transfer by minimizing the bending moments in the pile. In addition the pull-out resistance is significantly increased over an arrangement with the termination at the mudline. If the mooring lines are paired so that a single clumpweight is used for two lines then the installation tolerance required to ensure that both lines are evenly loaded requires that an anchor template be positioned on the ocean floor (4). Anchor piles are then driven through this template. In this arrangement the anchor line termination is at the mudline which would create a greater demand on a single pile; however, multiple piles may be used to anchor each line since the anchor template will serve to distribute each line load to more than one pile. Because of the large mean tension level in the anchor lines which will be sustained throughout the design life of the structure the long term effects of lateral creep in the soil must be considered in the design of the anchor piles.

The elevation at which the guyline enters the tower is chosen based on installation and operational considerations and is kept sufficiently below the water surface to avoid interference with service vessels. In Reference 1 the optimum location is recommended to be the level of the centroid of the design wave loadings on the structure. The guyline as shown in Figure 2-6 actually consists of two sections. The first section extends from the clumpweight to an outboard connection located several hundred feet away from the tower. The second section sometimes referred to as the pendant section extends from the outboard connection to the entry elevation. From the entry elevation, the pendant is directed through the tower using fairleads or hawse pipes to an elevation where the tensioning and holding devices are located. Either chain or wire rope may be used for the pendant section. In both cases the abrasion, corrosion and bending fatigue are important design considerations for the pendant section.

2.7.1 Behavior of a Single Mooring Line

In order to design an efficient mooring system a thorough knowledge and understanding of the design parameters which influence the mooring behavior is required. Fundamental to the array behavior is the behavior of a single mooring line. Traditionally in catenary spread mooring systems the design approach has been to increase the unit weight and length of the mooring lines until an acceptable combination is found. As will be shown below the guyed tower type mooring is a more complicated system than the conventional catenary mooring, and additional design parameters other than the unit weight and the length of the mooring line are available for use in its design. The following parameters govern the load deflection behavior of a single mooring line.

1. The pretension applied to the line.
2. The clumpweight intensity or weight per unit length.
3. The total weight of the clumpweight.
4. The angle of inclination of the guyline or in other words the distance the clumpweight is placed away from the tower.
5. The length of the anchor line.

The length of the anchor line contributes to the mooring line behavior in two ways. First, under extreme loading conditions, excessive mooring line tensions are likely to occur unless the line behavior becomes soft. The length of the anchor line governs the range of the deflections over which the soft behavior of the mooring line will be seen after the clumpweight has lifted. Once all of the anchor line is lifted off the sea floor, the mooring line tensions will begin to increase rapidly. The possibility of overloading a mooring line is reduced by making the anchor line longer. However under moderate tensions when the clumpweight is still partially resting on the sea bottom, the anchor line is fully supported and behaves as a lateral spring support for the mooring line. It therefore reduces the overall stiffness of the mooring line. Since the spring value of the anchor line is reduced as its length increases, the length of the anchor line should be no longer than necessary. A minimum length of one water depth plus several hundred feet is required for installation purposes so that the anchor system may be lowered separate from the clumpweight. In the designs to date this minimum length has provided a sufficient soft behavior range.

The lateral stiffness of a single mooring line consisting of an anchor line, distributed clumpweight and guyline is shown in Figure 2-7 for various values of clumpweight intensity and angles of inclination measured from the sea floor. The cable chosen for the guyline and anchor line in this example is 5 inch diameter wire rope, and the point of attachment to the tower is 1500 feet from the sea bottom. The length of the anchor line is 1800 feet. The stiffness values shown in this figure assume that the guyline is very taut and thus are the maximum values which may be expected for a 5 inch cable system.

In Figure 2-7 it is seen that the stiffness of the mooring line increases as the intensity of the clumpweight increases and as the angle of inclination decreases. However, note that the length of the guyline increases rapidly as the angle of inclination decreases below the 25 to 30 degree range. Practical limits for the minimum angle of inclination may be obtained from the maximum available length of a particular cable and the relative cost of handling a higher intensity clumpweight versus purchasing additional lengths of wire rope. The intensity of the clumpweight is limited by the allowable soil bearing pressures and the maximum desired width of the clumpweight. A first guess for possible ranges of the clumpweight intensity and the angle of inclination which provide a given initial stiffness for each individual mooring line may be obtained using Figure 2-7.

A third parameter which governs the load-deflection behavior of the mooring line is the magnitude of the pretension applied to the mooring line. The lateral stiffness of the guyline segment alone is shown as a function of the guyline tension in Figure 2-8. Curves for a wide range of angles of inclination are given. In Figure 2-8 the stiffness of the guyline is normalized with respect to the taut line stiffness value which was assumed when computing the curves for anchor line - clumpweight - guyline stiffness given in Figure 2-7. The taut line lateral stiffness may be found from the simple expression

$$K_T = \frac{AE}{L} \cos^2 \theta_b = \frac{AE}{Z} \cos^2 \theta_b \sin \theta_b$$

where AE is the equivalent rope modulus, L is the approximate length of the guyline and θ_b is the angle of inclination with respect to the sea bottom.

In Figure 2-8 one may note that as the angle of inclination decreases larger percentages of the breaking strength of the rope are required to achieve the same ratio of actual guyline stiffness to the taut line stiffness. Furthermore, for the 5 inch wire rope and water depth considered in this example, to actually achieve the taut line stiffness value requires tensions around the breaking strength of the rope. Therefore the effect of the guyline tension must be included when attempting to predict the stiffness of a guyed tower mooring line particularly for the smaller values of the angle of inclination.

The initial stiffness of a single mooring line with an angle of inclination of 25 degrees is given in Figure 2-9. In this figure the single line stiffness is plotted as a function of the clumpweight intensity for various ratios of pretension to the rope breaking strength. For a given value of pretension, the stiffness of the anchor line - clumpweight - guyline system increases as the clumpweight intensity increases up to a limiting value determined primarily by the stiffness of the guyline section.

In the following figure, Figure 2-10, the value of the pretension has been set equal to 30 percent of the breaking strength of the guyline. The initial stiffness of the mooring line is plotted as a function of clumpweight intensity and angles of inclination. Similar curves could be drawn for other values of pretension with higher pretensions generally producing larger values of initial stiffness. One point to note in comparing possible combinations of pretension and angle of inclination is that larger pretensions and angles of inclination will produce higher amounts of vertical load on the platform. This aspect will be discussed further in a later section.

2.7.2 Behavior of the Mooring System

Thus far the discussion has primarily centered around the estimation of the initial stiffness of an individual mooring line. However in the design requirements the known desired stiffness quantity is the overall stiffness of the entire mooring system. In order to determine the requirement for a single mooring line, a qualitative understanding of the behavior of the mooring array in terms of the individual mooring lines is required. Without going into elaborate details consider the simple example of two opposing mooring lines shown in Figure 2-11.

The instantaneous lateral stiffness of the two line system at a particular value of deflection is equal to the sum of the individual stiffness of each line and is a function of the instantaneous tension of each line. For example, consider that both lines are tensioned very highly, then the initial stiffness of the array would be twice the limiting stiffness of an individual line. As the tower deflects to the right increased tension in the left side line will not produce any larger stiffness, since it is already at the limit, and the decreasing tension on the right side will reduce the stiffness of that line eventually to a very low value. Therefore, the array exhibits a softening behavior with the limiting array stiffness for the deflected tower being equal to the limiting stiffness for one line which is one-half of its initial value.

Now consider that both lines have a very low value of pretension such that the initial stiffness of each line is zero for pratical purposes. As the tower deflects the line on the left side will tighten, and its stiffness will increase eventually to its limit if the deflection is great enough. The line on the right side will remain slack and will not contribute to the array stiffness. Therefore the limit of the array stiffness for the deflected tower is again equal to that of a single line, and the array shows a hardening behavior. It stands to reason that for some moderate tension, between the very taut and the very slack cases, the combination of the two lines will produce a combined stiffness which varies only slightly about the limit for a single line as shown in Figure 2-11. It should be noted that the softening or hardening behavior discussed above is due to the mooring line pretension and is seen in the initial portion of the mooring array response curves.

If the angle between each mooring line and the direction of deflection is denoted by α_i and the instantaneous stiffness of each line in its radial direction is k_i then the instantaneous array stiffness is given by

$$K_I = \sum_{i=1}^{N} k_i \cos^2\alpha_i$$

where N is the number of mooring lines.

Assuming that all the lines are pretensioned identically the stiffness of the array at zero tower offset for the 24 cable array shown in Figures 2-4 and 2-5 is found to be twelve times the stiffness of a single line. As an example consider that the array stiffness sought is 275 kip/ft. then the required initial single mooring line stiffness is 23 kip/ft, assuming a quasi-linear response for the opposing line pairs. From the data given in Figures 2-9 and 2-10, one choice for a 5 inch mooring line configuration would be an angle of inclination of 25 degrees, a clumpweight intensity of 3 kip/ft and a pretension of 30 percent of the breaking strength of the cable.

As a first check of the behavior of the above mooring line as the tension changes recall the data for the guyline section given in Figure 2-8. For an angle of inclination of 25 degrees and pretension of 30 percent of the breaking strength (T_u), the initial stiffness of the guyline section is approximately 0.5 of the limiting value (k_{max}). If the tension increases to 50 percent of T_u the stiffness increases to approximately 0.85 k_{max}, while if the tension decreases to around 20 percent T_u the stiffness decreases to about 0.2 k_{max}. Therefore as the tower deflects the opposing lines should interact to maintain the stiffness within a reasonable amount of the initial value.

As a further check of the choice of mooring line parameters before proceeding with an actual analysis, the data given in Figure 2-9 may be replotted as shown in Figure 2-12. In this figure the horizontal stiffness of the 5 inch mooring system has been plotted versus the maximum line tension for a single value of clumpweight intensity, the chosen 3 kip/ft value. As seen in this curve the starting tension of 30 percent T_u produces the desired single line stiffness of 23 kip/ft. At a maximum line tension of 50 percent of the breaking strength the stiffness has increased to 30 kip/ft. The tower offset which will produce a change in tension from 30 to 50 percent is estimated to be 20 ft. For a 20 ft. deflection the tension in the slacking line is estimated to be 19 percent T_u with a corresponding stiffness of 12 kip/ft. If only two opposing lines are considered the combined initial stiffness is 46 kip/ft which will result in an initial array stiffness of 275 kip/ft. As the tower deflects until the line with increasing tension reaches 50 percent T_u, the instantaneous stiffness of the two lines decreases to 42 kip/ft. Therefore the array will be slightly softening; however, the choice of mooring line still appears feasible.

Thus far we have defined the size of the cables (5 inch), the angle of inclination (25°), the length of the anchor line (1800 ft.), and the intensity of the clumpweight (3 kip/ft). The remaining quantity to be defined is the total weight or length of the clumpweight. Two factors influencing this choice are (1) the maximum allowable tension and (2) the required strength level of the mooring system.

The total weight of the clumpweight to limit the tension to a value of T_m may be estimated using the formula given below,

$$W = \sqrt{(T_m - q_G Z)^2 - (T_m - \frac{q_G Z}{2}) \cos^2 \theta_b}$$

where q_G is the submerged unit weight of the guyline and Z is the vertical projection of the guyline. In most cases Z may be taken to be equal to the distance from the tower fairlead to the ocean floor since the height of the upper end of the clumpweight above the sea floor is usually small compared to the depth.

For a maximum line tension of 1450 Kips, which is one-half the breaking strength of the 5 in. rope, the weight of the clumpweight would be on the order of 510 Kips resulting in a length of 170 ft. for the clumpweight. Lifting of the clumpweight will occur at a tower offset of 19 ft. for a 510 kip clumpweight. Due to the strength requirements of the tower for which this mooring was designed it was found that the size of the clumpweight could be reduced somewhat. The length of the clumpweight was reduced to 157 ft. with a total weight of 471 Kips. Total lift will then occur at a tower of offset of 16 ft. and a maximum line tension of 47 percent of the breaking strength.

The design of the mooring line thus far has been accomplished by using approximate relationships for the line stiffness and a few special solutions such as the solution for the tension and offset at the point of total lift of the clumpweight (9). It now remains to perform the mooring analysis which will determine the actual static mooring response and verify the design. For purposes of this discussion, only dead weight forces are considered to act along the cables. The variation of cable tension as a function of the tower movement is shown in Figure 2-13.

The horizontal force required to move the system for various values of horizontal displacement is shown plotted in Figure 2-14. The single line mooring exhibits a hardening behavior for deflections up to about 16 ft. and then the stiffness of the line begins to reduce due to the lifting of the clumpweight. The relative position of the clumpweight for various values of tower movement is shown in Figure 2-15. Initially, for no lateral deflection of the tower the front portion of the clumpweight is off the sea bottom. About 80 percent of the clumpweight is off the sea bottom when the tower moves 10 ft. laterally, and finally full lift of the clumpweight occurs for tower movements greater than 16 ft.

The discussion of the mooring system's behavior thus far has assumed that no forces are acting which would restrain the lifting of the clumpweight. These forces, called suction forces, will develop if the clumpweight becomes embedded in the sea floor due to settmelent or other factors and should be considered in the design of the clumpweight (10). If the clumpweight were a single concentrated weight, soil suction could result in a significant increase in the predicted maximum line tension since the vertical force needed to lift the clumpweight off the bottom would increase by an amount equal to the suction forces. In contrast very little difference is found in the predicted tensions for mooring lines using distributed clumpweights when suction is included since the suction forces act at discrete points along the distributed clumpweight. Due to the lifting action of the distributed clumpweight only one of the discrete suction forces must be overcome at any one time. The value of the discrete suction force depends on the clumpweight geometry along its length and the rate of pull; however, these forces are generally small when compared to the total weight of the clumpweight.

The force-deflection behavior of a mooring system consisting of 24 guylines is shown in Figure 2-16. The ordinate in this figure shows the horizontal force necessary to produce the corresponding lateral deflections shown on the abscissa. Note that the force-deformation behavior of the mooring system is essentially linear up to a deflection of about 16 ft. For larger deflections, the clumpweights begin to lift off from sea bottom resulting in a softening of the force-deflection behavior. There is a smooth transition between the stiff and soft regions of the curve; however, the general nature of the curve is such that a bilinear representation is adequate for preliminary design studies of the tower.

An undesirable aspect of the mooring system is the vertical component of the cable reactions. These forces are transferred to the tower at the point of attachment of the cables, and the foundation system must be designed to carry these loads. The vertical force versus the horizontal deflection of the tower is shown in Figure 2-17. It may be noted that the vertical component of cable reaction is a significant design parameter, but the change in vertical force due to tower movement is not significant. This means that the vertical cable reaction is primarily governed by the pretension with higher pretensions resulting in larger vertical forces. Thus the optimum mooring system is the one which minimizes the vertical component of cable reaction, maximizes initial stiffness and has the desired strength level providing an adequate factor of safety against failure under predicted values of tower deflections produced by the design storm.

Another important design consideration is the behavior of the mooring system under damaged conditions. Assuming that two adjacent mooring lines are damaged, the force-deflection behavior of the system is shown in Figures 2-16 and 2-17. Since the lines diametrically opposite to the damaged lines are intact, nonzero initial deflection is noted, and both the initial and final stiffnesses of the mooring system are reduced. The platform behavior should be investigated for the damaged condition of the mooring system to ensure acceptable platform response.

2.8 DESIGN OF TETHER MOORING SYSTEM

Design considerations for the second class of mooring systems referred to as a tether system are presented in this section. In the tether type mooring the line is suspended between the tower and anchoring system as shown in Figure 2-18. The anchoring system supports the tether both horizontally and vertically. The tether type system has an obvious economic attractiveness in that it eliminates the material and installation costs associated with the clumpweights and anchor lines. The design of a tether system using wire rope is discussed below. Designs using synthetic ropes are also possible but will not be considered in this report.

One approach to begin the design of a tether system without clumpweights and anchor lines is to anchor the guyline section to the sea floor at the location of the clumpweight as shown in Figure 2-18. But such a system will result in excessive tensions in the cable under design level deflections of the tower.

The stiffness of a 5 in. wire rope with a pretension of 30 percent T_u is 38 kip/ft which is greater than the required 23 kip/ft for a 24 cable array. Therefore it is possible to increase the flexibility of the tether line and obtain the required stiffness. Three possible actions which will increase the tether flexibility are (1) lower the pretension which will introduce additional sag, (2) place the anchor position further from platform which will increase the cable length, and (3) select a different rope construction which has a lower rope modulus.

For the particular cable under consideration it was found that changing only the pretension was required. The force deflection behavior of a pair of opposing lines is shown in Figure 2-19 for various values of pretension. The force-deflection behavior for a mooring system consisting of 24 lines pretensioned to 20 percent of their breaking strength is shown in Figure 2-20. On the same figure is shown the force-deflection behavior of the mooring system with clumpweights. The behavior of both mooring systems is essentially the same for small values of tower deflections. However the behavior of the two systems is very different for the larger values of the deflections. The system with clumpweights and anchor lines exhibits a softening behavior. As the tower deflection increases the stiffness of the mooring system decreases. The implications of this softening behavior are that the apparent natural period of the tower increases and that the tension in the individual line does not increase with tower deflections. The clumpweight acts as a safety valve which limits the tension in the cable and the resultant forces on the tower. On the other hand, the mooring system without clumpweights exhibits a hardening behavior since as the tower deflection increases the stiffness also increases. The instantaneous value of the tower period shortens and both the tension in the cable, and the resultant tower forces increase at a faster rate than for small deflections.

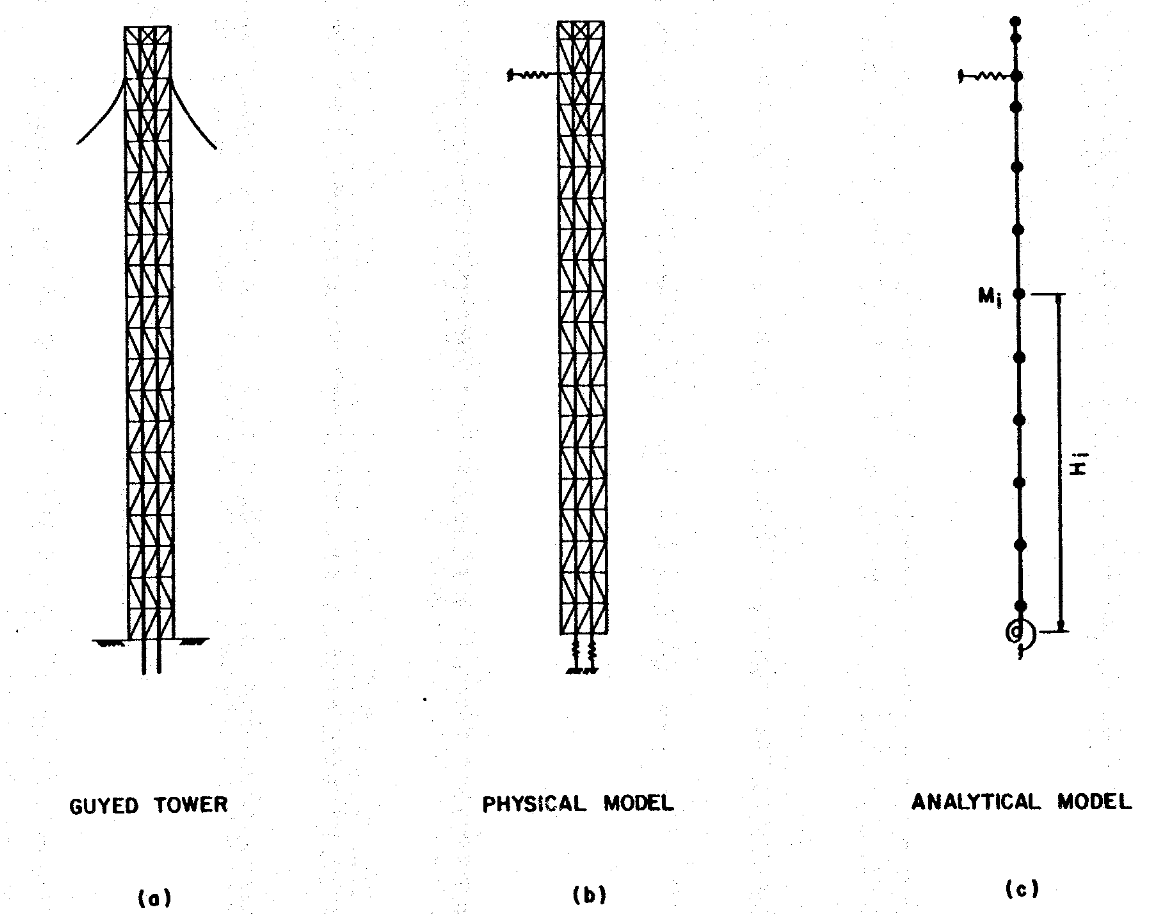


FIGURE 2-1
ANALYTICAL MODEL

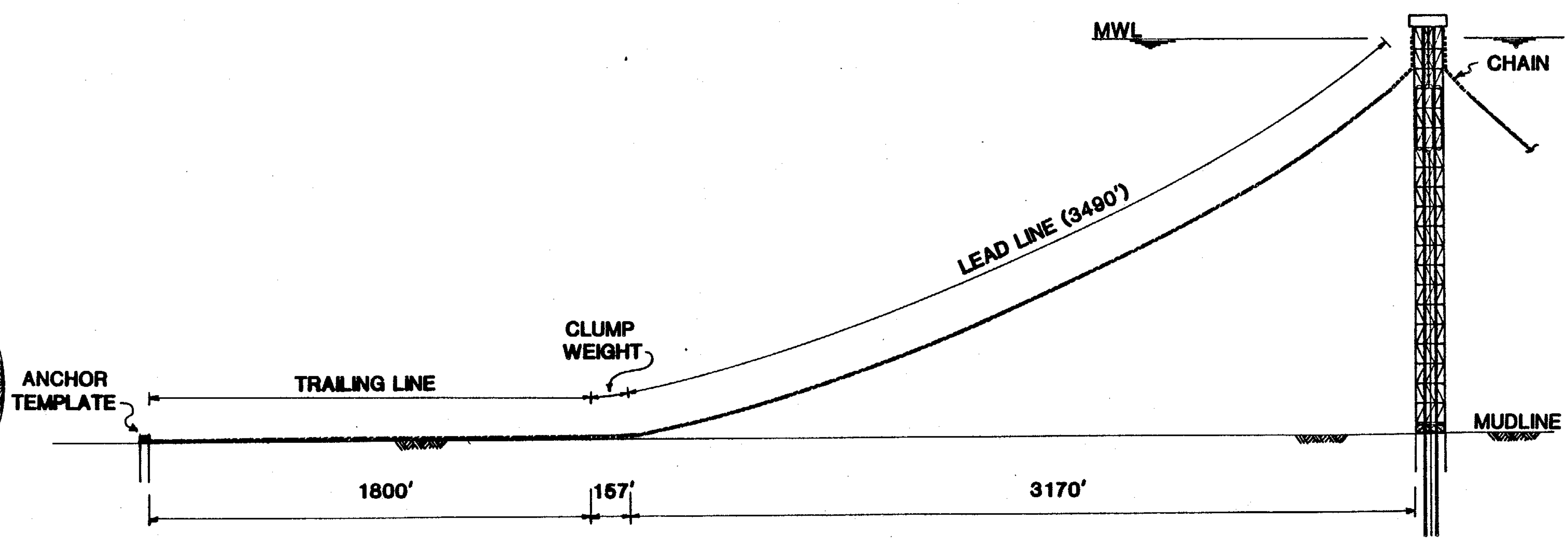
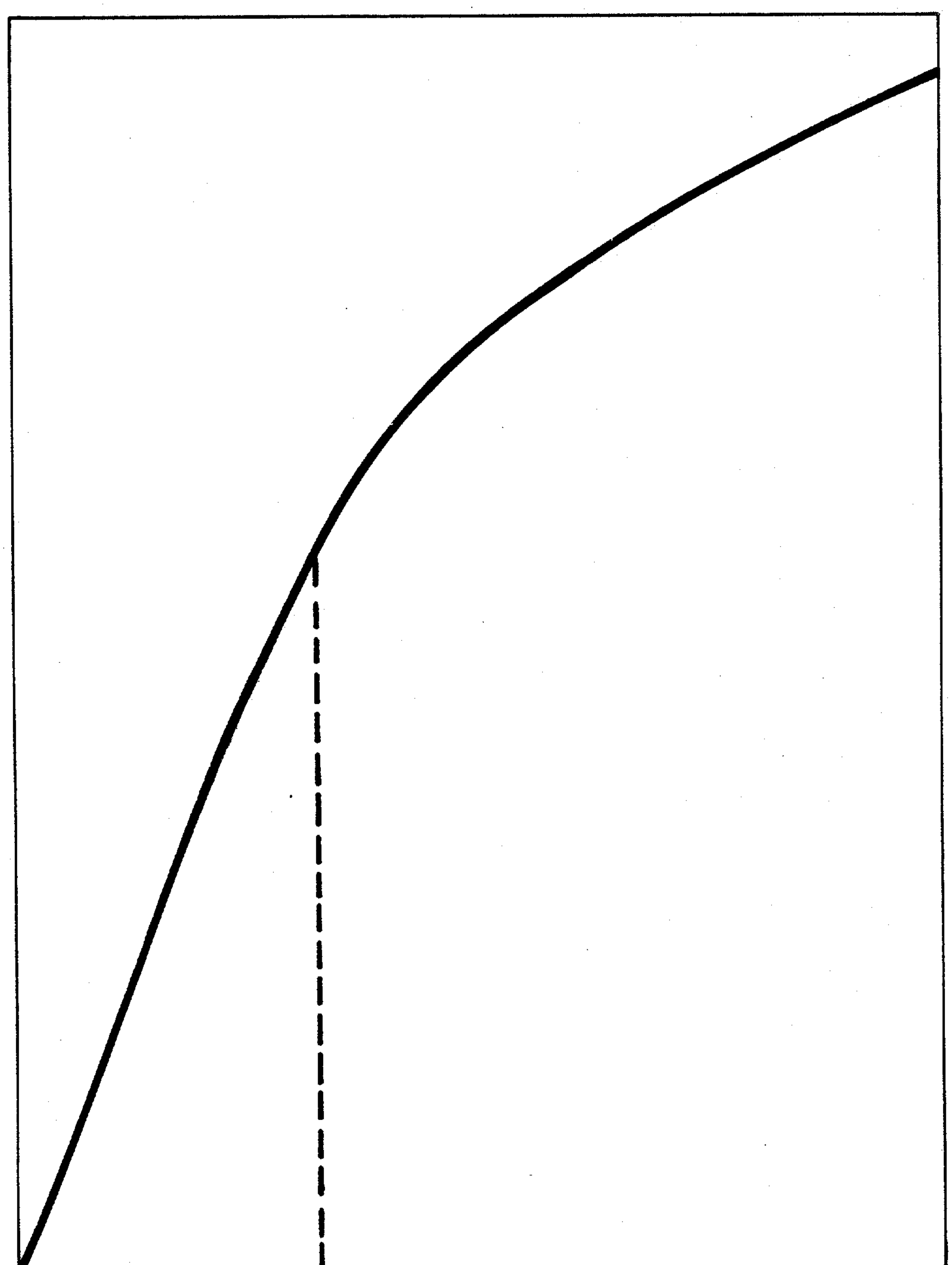


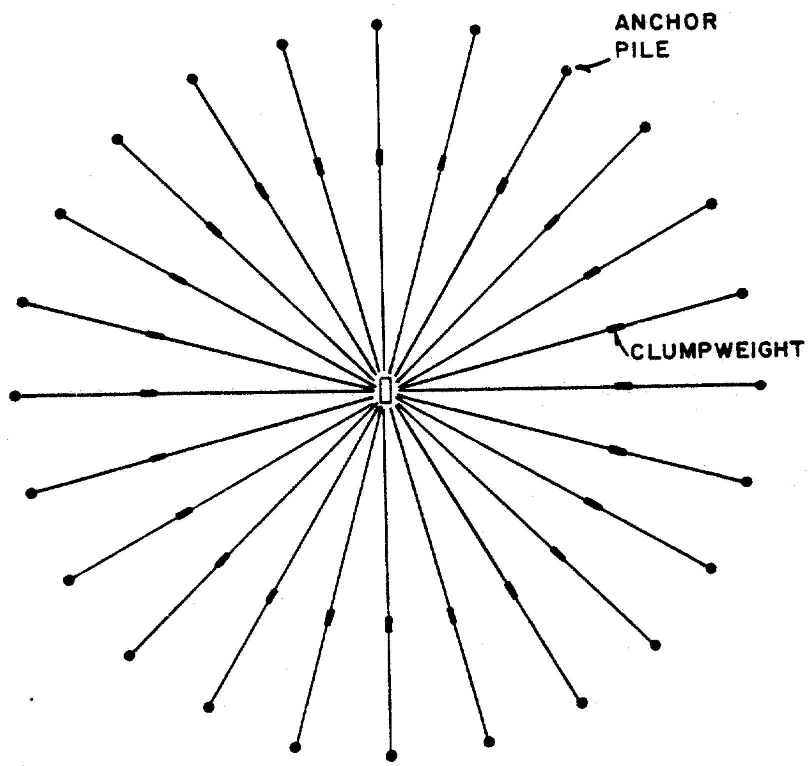
FIGURE 2-2

GUYED TOWER MOORING SYSTEM WITH CLUMP WEIGHTS



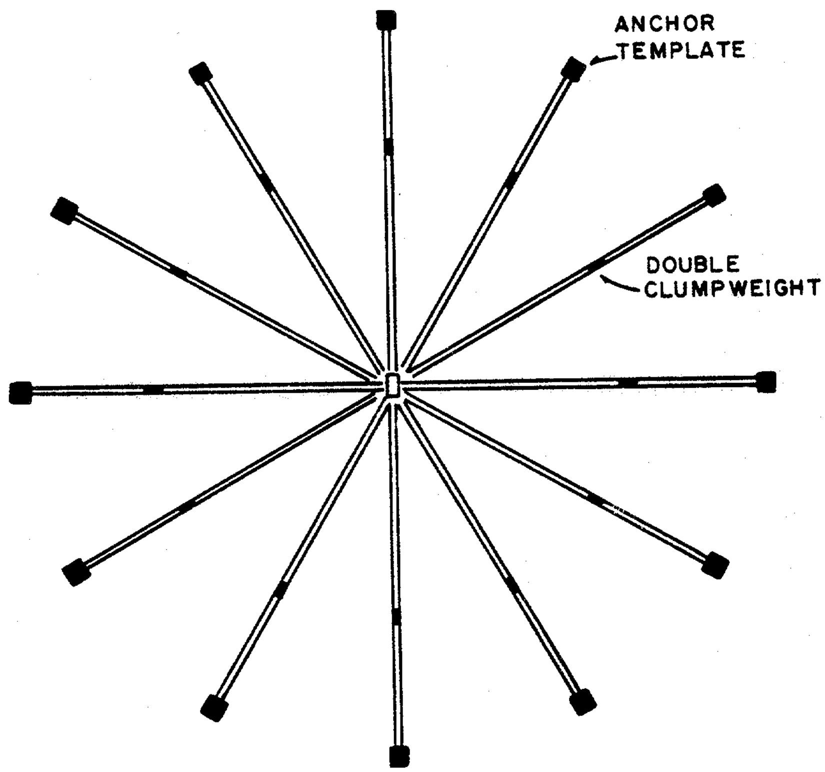
MOORING SYSTEM BEHAVIOR

FIGURE 2-3



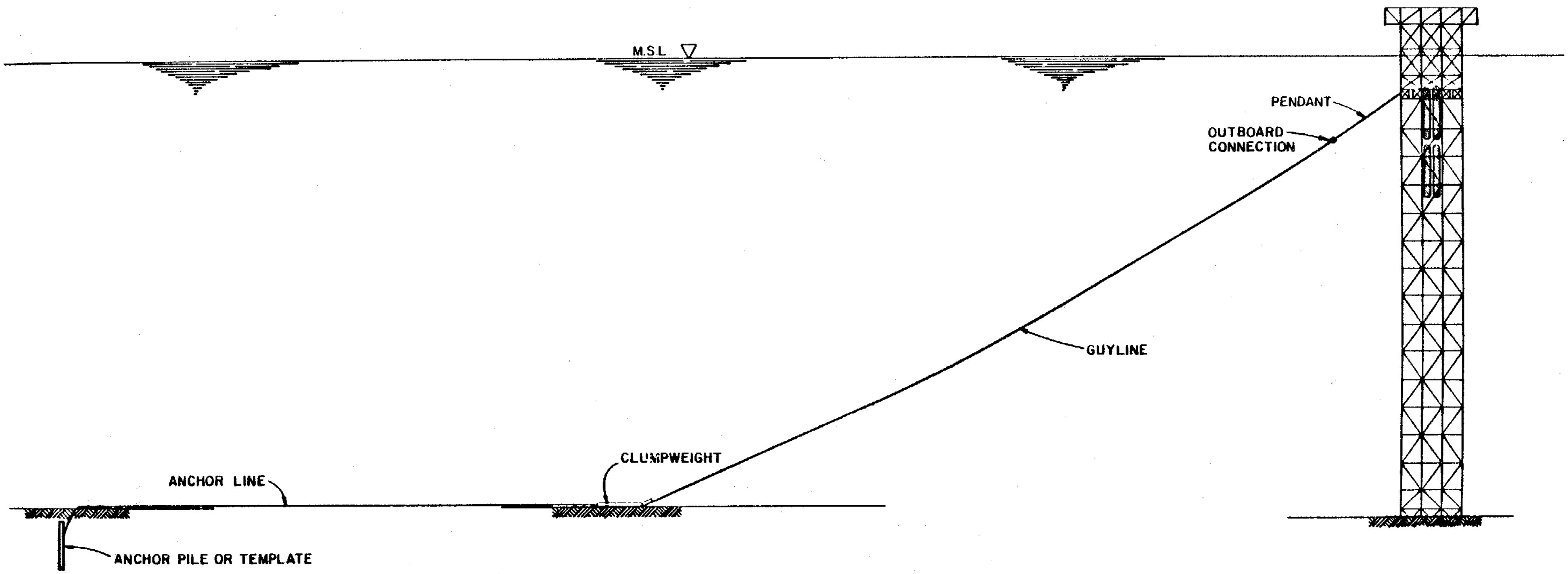
TWENTY FOUR MOORING LINE ARRAY
SINGLE LINE ARRANGEMENT

FIGURE 2-4



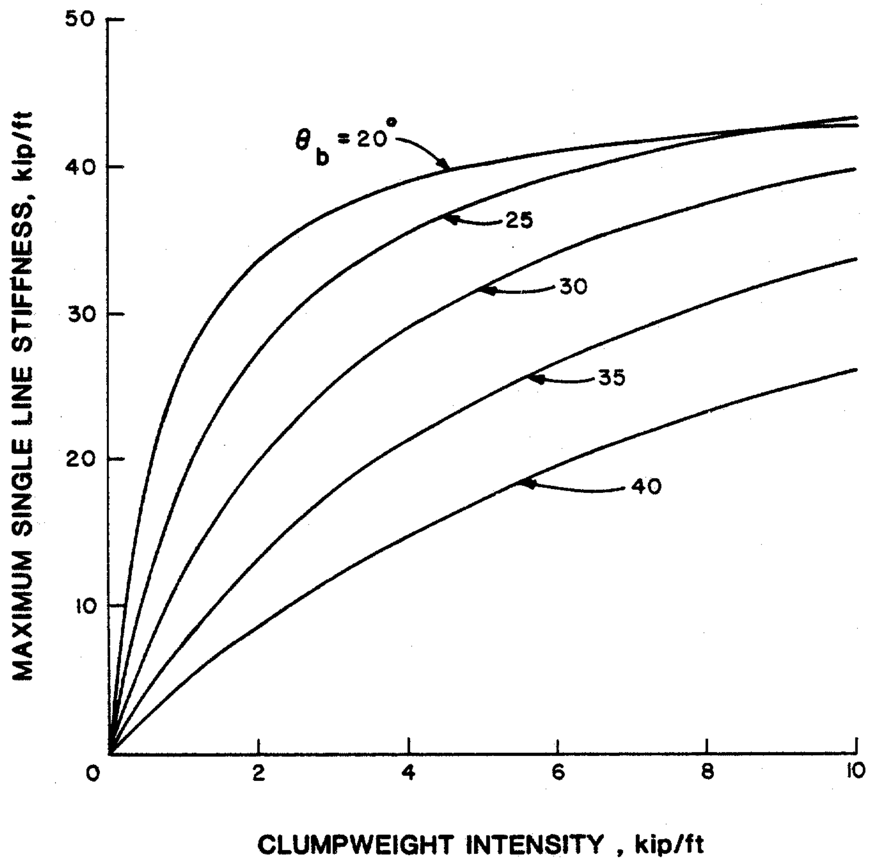
TWENTY FOUR MOORING LINE ARRAY
PAIRED LINE ARRANGEMENT

FIGURE 2-5



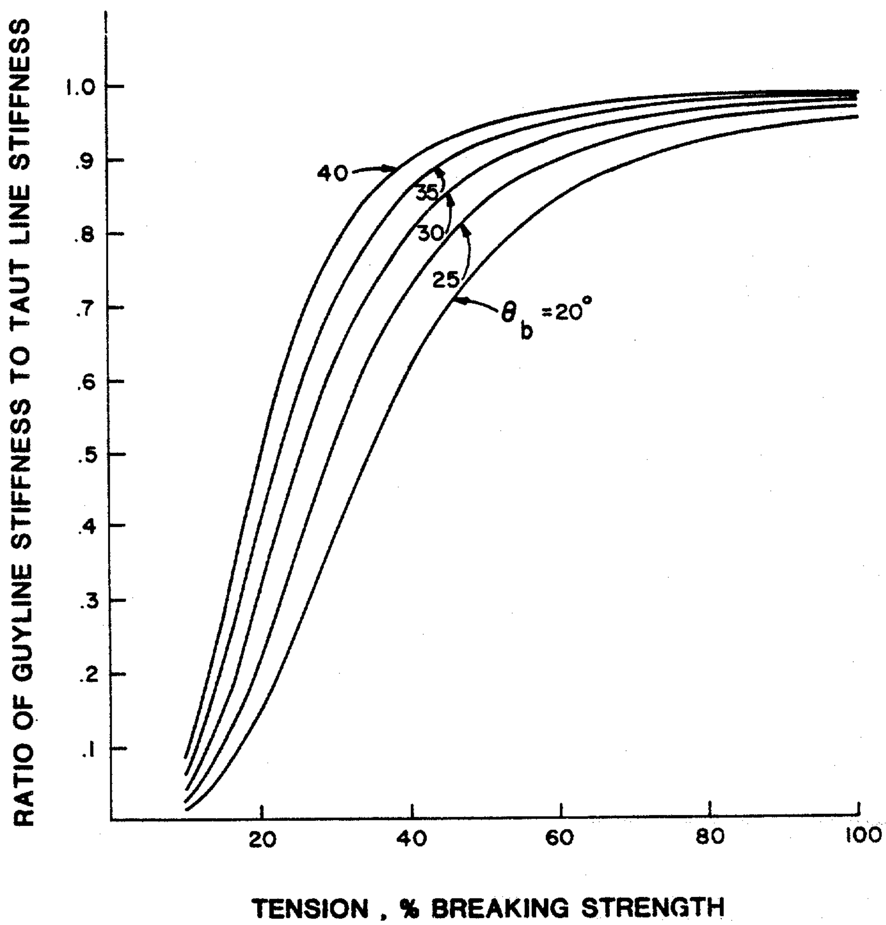
ELEVATION VIEW OF A TYPICAL GUYED TOWER MOORING

FIGURE 2-6



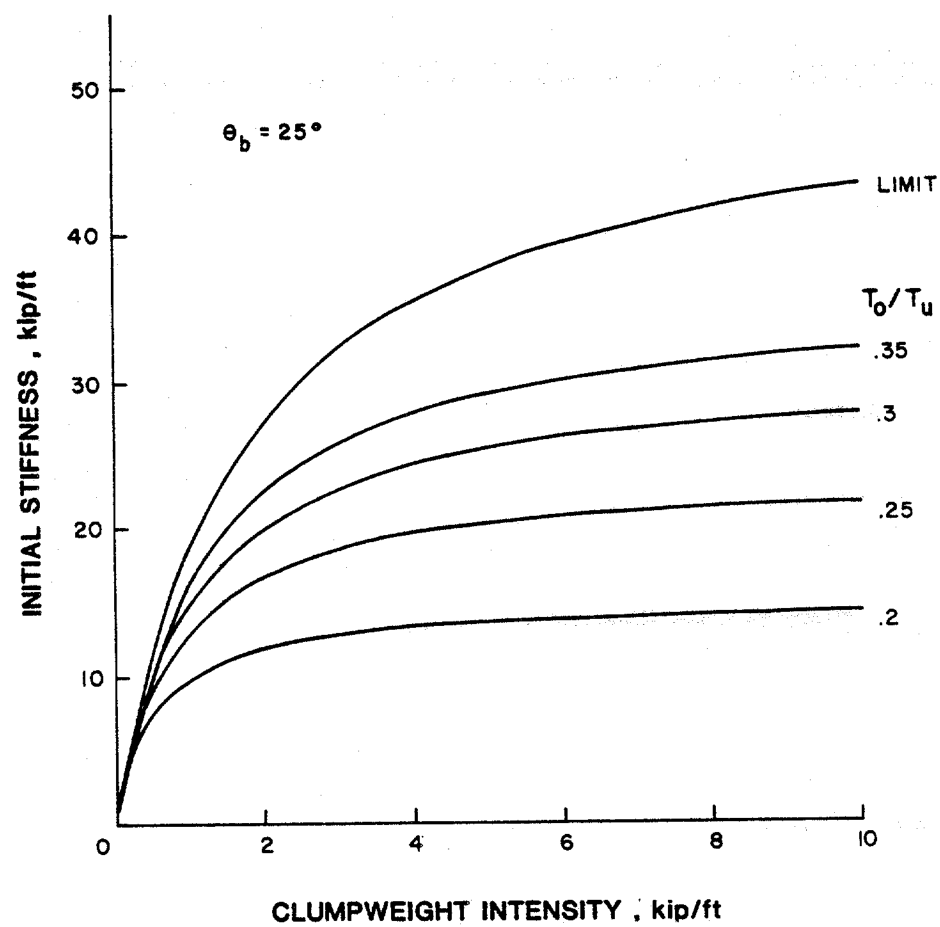
MAXIMUM STIFFNESS OF AN ANCHOR LINE-CLUMPWEIGHT-GUYLINE SYSTEM

FIGURE 2-7



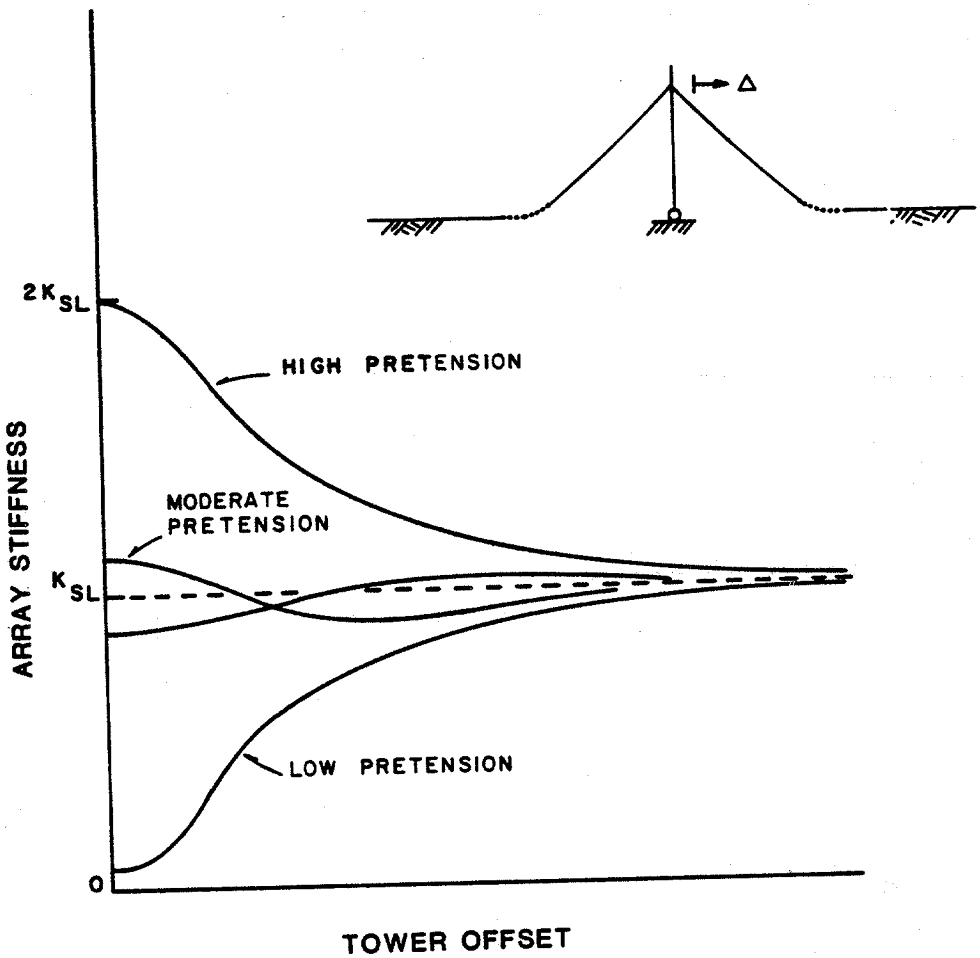
EFFECT OF TENSION ON GUYLINE STIFFNESS

FIGURE 2-8



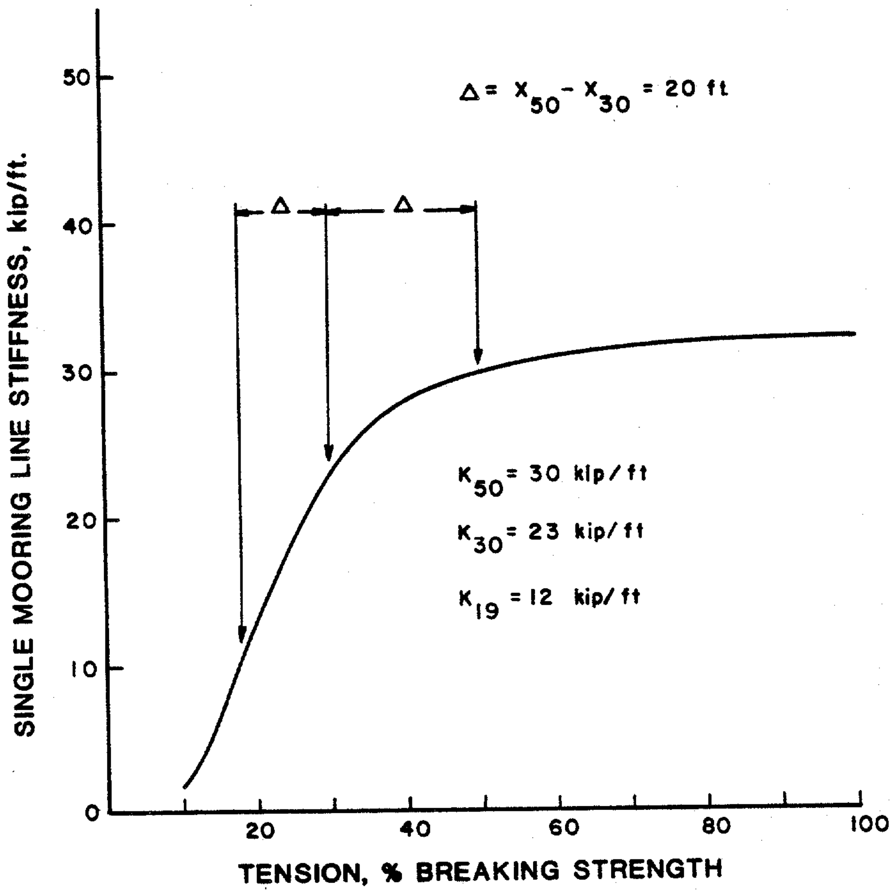
EFFECT OF PRETENSION ON THE INITIAL SINGLE LINE STIFFNESS

FIGURE 2-9



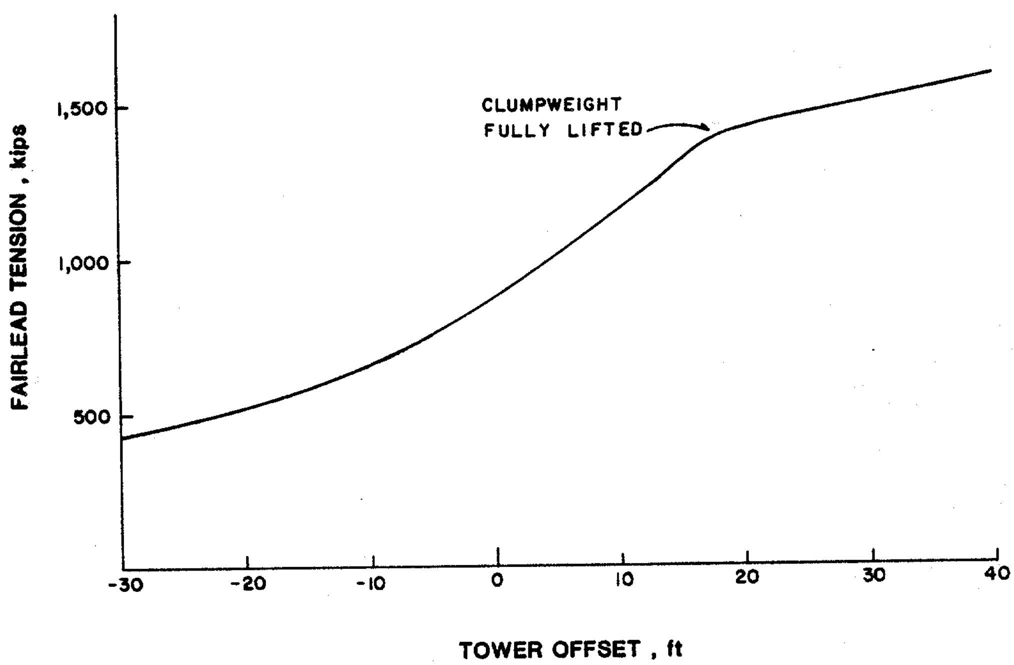
EFFECT OF PRETENSION ON ARRAY BEHAVIOR

FIGURE 2-11



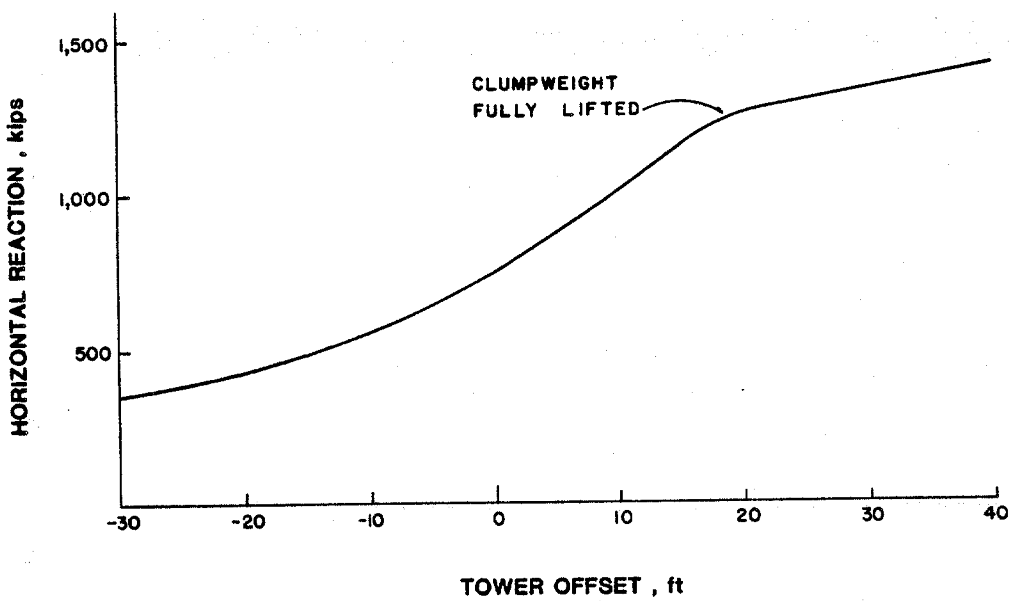
MOORING LINE STIFFNESS AS A FUNCTION OF THE GUYLINE TENSION

FIGURE 2-12



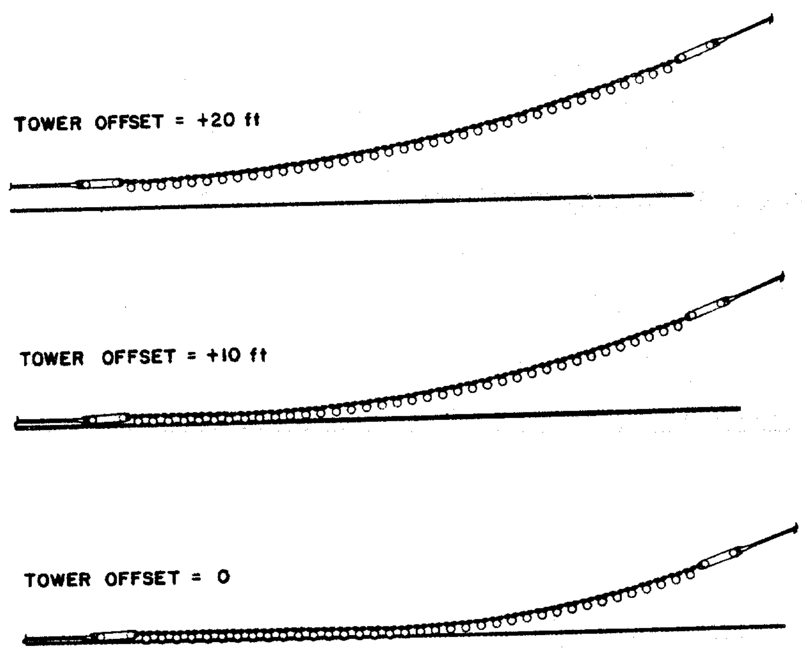
MAXIMUM SINGLE LINE TENSION

FIGURE 2-13



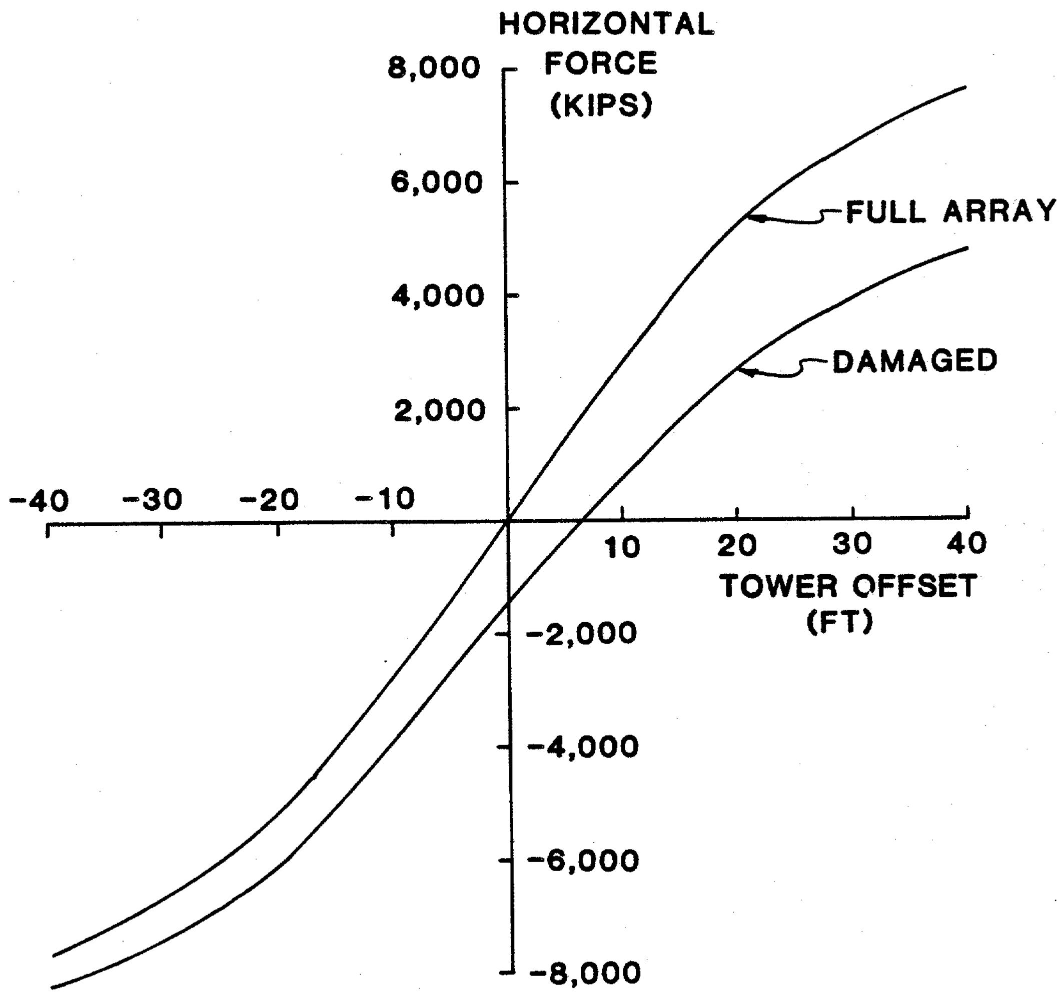
SINGLE LINE HORIZONTAL REACTION

FIGURE 2-14



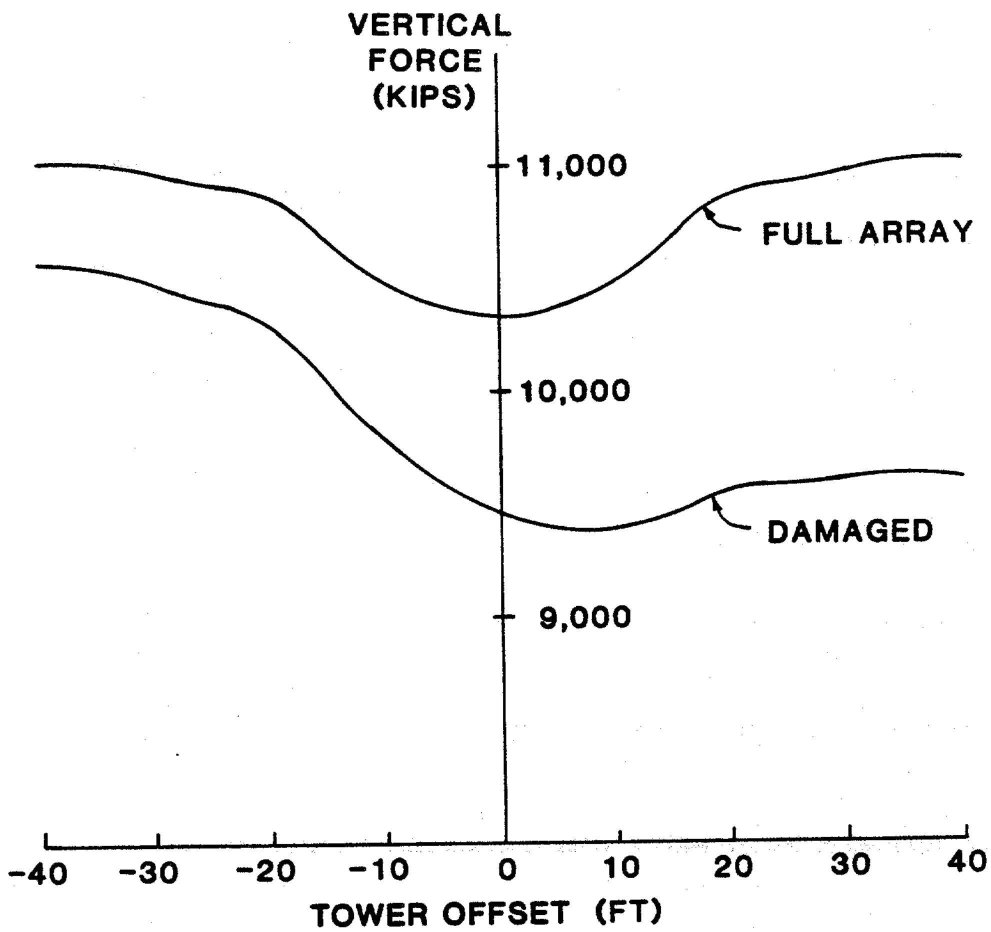
CLUMPWEIGHT PROFILE AT VARIOUS TOWER OFFSETS

FIGURE 2-15



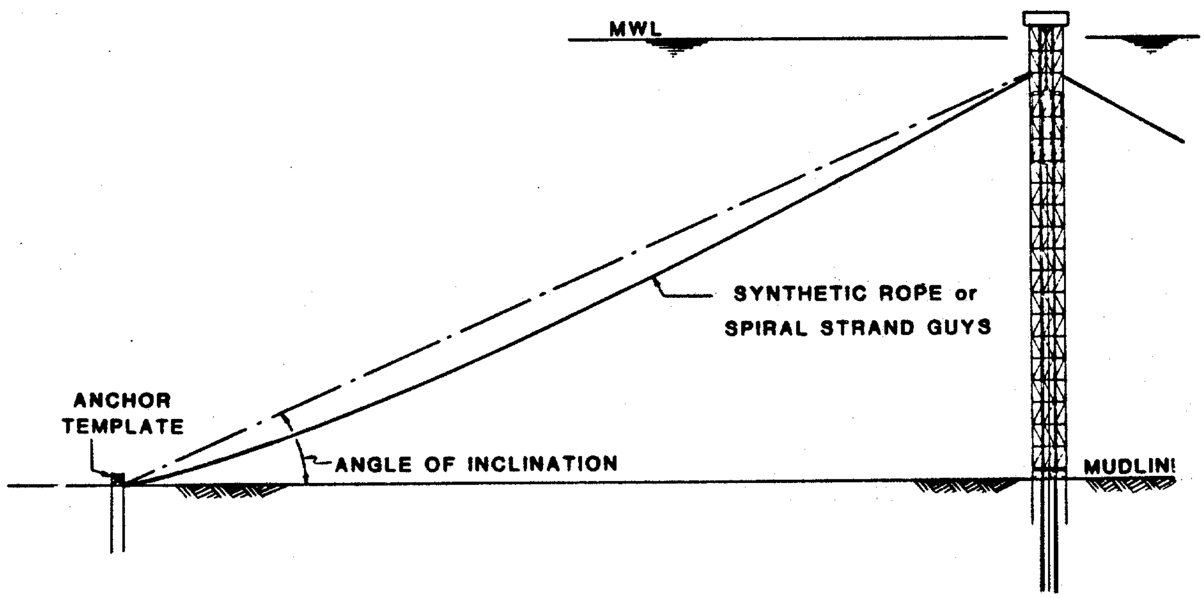
HORIZONTAL ARRAY REACTION

FIGURE 2-16



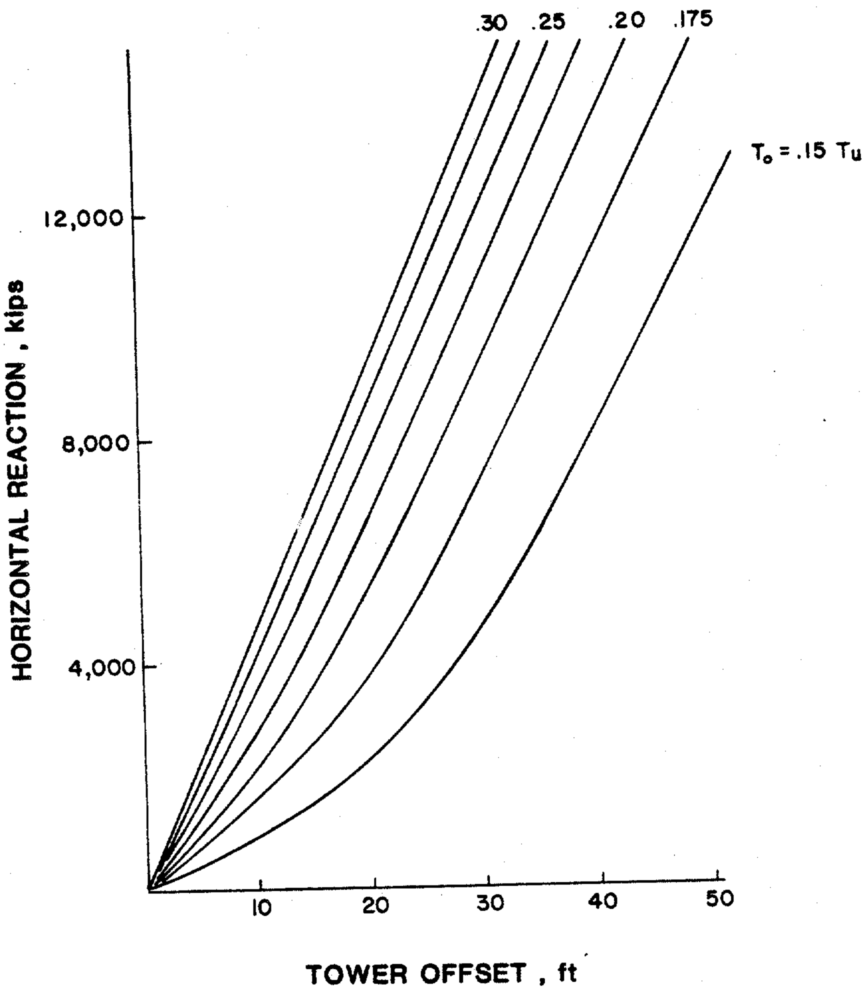
VERTICAL ARRAY REACTION

FIGURE 2-17



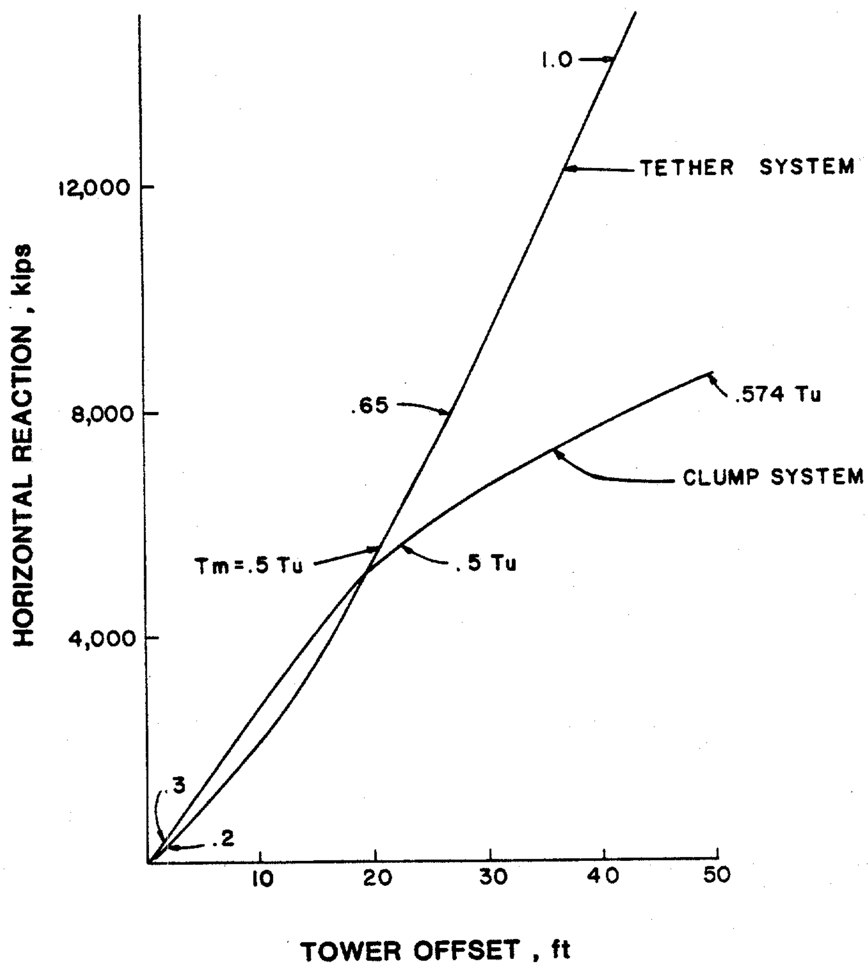
ELEVATION VIEW OF A TETHER MOORING LINE

FIGURE 2-18



EFFECT OF PRETENSION ON TETHER ARRAY RESPONSE

FIGURE 2-19



COMPARISON OF TETHER AND CLUMPWEIGHT SYSTEMS

FIGURE 2-20

3. ANALYTICAL ASPECTS

3.1 GUYED TOWER MODELLING

The specific details of the guyed tower analytical model will depend upon the particular manner in which the results are utilized and the accuracy of the input data. Simplified models will suffice to predict overall responses, especially in preliminary design stages and during parametric studies. However, more refined models will be required during final design stages.

The guyed tower model should reflect the key analytical parameters of stiffness, mass, and damping. The stiffness should include the stiffness of the tower-deck system, stiffness of the mooring system and appropriate idealization of the foundation. Both the mooring and foundation systems will require special modelling considerations.

The overall response of the guyed tower can be predicted using a simplified representation of the mooring system. The mooring system can be idealized as a massless spring having nonlinear force displacement behavior derived from a static analysis of the mooring system. Figures 3-1 and 3-2 show the force-deflection behavior of the horizontal and vertical springs to be used in the overall platform model. Note the vertical ordinate corresponding to zero tower movement in Figure 3-2. This is due to the pretension in the individual mooring lines and should be included in all inplace analyses.

Similarly a simplified representation of the foundation system will suffice in most analyses performed to obtain overall platform behavior. Modelling considerations for spud can foundations is discussed in Ref. 11. Linearization of the pile foundation behavior is routinely performed to predict the overall response of fixed platforms. Such linearization when performed at the level of response expected under the particular loading conditions is adequate for design applications.

Besides the elastic stiffness of the tower, mooring system and the foundation, the guyed tower analytical model should also consider the geometric stiffness (large displacement) effect. Consideration of geometric stiffness will increase the natural period of the platform and produce second order P-Delta forces which contribute to the exciting forces.

The mass should include that of the platform steel, all appurtenances, conductors, deck loads, the mass of water enclosed in submerged tubular members, and the added mass of submerged members.

The major sources of damping in guyed towers are: (1) structural damping, (2) hydrodynamic damping, and (3) foundation damping. Since the guyed tower responds as a rigid body in its fundamental mode, structural damping may be ignored. Hydrodynamic damping originates from two sources: the radiation effects of propagating water waves and the relative velocity between the vibrating structure and fluid. The latter should be explicitly included in the analytical procedure. The energy dissipation in the soil includes two components: hysteretic energy dissipations due to cyclic loading that takes place mainly in the soil adjacent to the pile and the geometric or radiation energy dissipation that occurs due to the propagation of elastic stress waves away from the pile. References 12, 13 should be consulted for dynamic behavior of piles and 14 for spud can foundations.

The platform model used to compute environmental forces should accurately represent all major platform elements. Consideration of marine growth is particularly important in wave force computations.

Subsequent sections of this report will discuss two types of guyed tower models: a simplified model used to predict overall platform response, and a detailed model which is recommended in final design stages. The analytical procedures to be used are essentially the same for both models. Additional complexities are introduced in the coupled analysis of the tower-mooring system and in treating the foundation nonlinearities.

3.2 DYNAMICS OF GUYED TOWER

The fundamental properties which provide insight into the dynamic behavior of a structure responding linearly are its natural periods and associated mode shapes. Strictly speaking, guyed towers are not linear systems; however, their behavior can be linearized over a wide range of response parameters. The principal sources of nonlinearity in a guyed tower are the nonlinear load-deflection relationship of the mooring system and the nonlinear hysteretic behavior of the soil-pile system. The behavior of the array of cables is linear for small to moderate values of tower deflection (Figure 2-3). Soil behavior is nonlinear over a wide range of response conditions. However, linearization of the foundation stiffness is routinely performed to predict the overall response of fixed platforms. Such linearization when performed at the level of response expected under the particular loading conditions is adequate for design applications.

The dynamic behavior of a guyed tower platform subjected to a wave excitation is governed by three types of modes of vibration. These are the sway, flexural and torsional modes. The sway mode is the fundamental mode in a particular lateral direction and is basically a rigid body mode with little or no bending. The natural period of this mode is governed by the height of the tower, the magnitude and distribution of the mass and above all by the lateral stiffness of the mooring system. Under wave excitations the tower movements are controlled by the sway mode. In typical guyed tower designs the period of the sway mode is about twice the predominant period of the design sea state.

The second mode in the same lateral direction is a bending (flexural) mode. The period and the associated mode shape of the second mode are primarily governed by the magnitude and distribution of the mass and the stiffness of the tower as well as the lateral stiffness of the foundation. The stiffness of the mooring system has practically no effect on the flexural period of the guyed tower. A good approximation to the flexural period of the guyed tower can be obtained by idealizing it as a beam pinned at the base and free at the other end. However, the mode shape is strongly influenced by the lateral restraint provided by the foundation.

The properties of the flexural mode are important for the prediction of stresses and deformations in the tower. The tower should be proportioned such that its flexural period is much shorter than the predominant period of the design wave so that excessive stresses in the tower can be avoided. Another consideration is the fatigue behavior of the tower. Since fatigue is controlled by the smaller waves having shorter periods, a prudent approach is to design the tower to behave as a stiff structure in its flexural mode.

The third category of vibrational mode which governs the guyed tower design is torsion. The primary source of torsional stiffness is the foundation. The design should minimize the torsional period so that dynamic amplifications in torsional excitation can be avoided, especially under the frequently occuring smaller waves.

A fourth type of platform mode which is quite important for earthquake type excitations is the vertical mode. The vertical period is much less than the flexural and torsional periods. The first three modes of a guyed tower are shown in Figure 3-3.

Approximate Computation of Sway Period

Even though computerized procedures and softwares are available to compute the periods and mode shapes of complex structures, simplified procedures are valuable especially in early design stages. The natural period of the sway mode can be easily computed using the

Rayleigh method. The basis of this procedure is to assume a mode shape and then compute the maximum kinetic and potential energies under free vibration. By equating the two, the period of vibration can be computed. The mode shape can be assumed to be a straight line, with the base of the tower being pinned. Referring to Figure 2-1(c)

Let M_i = mass at the ith level
H_i = height of mass i from the base
K_c = cable stiffness
H_c = height of location of cable from the base

The period of the sway mode is computed from the relation (5)

$$T = 2\pi \sqrt{\frac{M_i H_i^2}{K_c H_c^2}}$$

The accuracy of the above expression is quite satisfactory for most preliminary applications. The natural period computed using the above relation will be smaller than the period computed by more accurate procedures, since the platform is constrained to vibrate in a particular shape in this approximation.

It is the usual practice in engineering computations to neglect the effect of the geometric stiffness when computing platform periods and mode shapes. Consideration of the geometric stiffness will result in a decrease in stiffness and hence an increase in period. This increase in period due to geometric stiffness is negligibly small for shallow water platforms. It is of some importance for the tall slender guyed towers. The sway period was found to increase by about 10 percent due to geometric stiffness for a guyed tower in a 1600 ft. water depth. The following simplified relation can be used to evaluate the effect of geometric stiffness on sway mode.

$$\frac{T}{T_o} = \frac{1}{\sqrt{1 - r}}$$

in which T_o = Period computed without the effect of geometric stiffness

T = Period considering geometric stiffness

$$r = \frac{W\,H}{K_c\,H_c^2}$$

W = Net vertical load on the platform

K_c = Cable stiffness

H_c = Height of the point of attachment of cable to the tower from the base.

3.3 GUYED TOWER BEHAVIOR UNDER WAVES

The behavior of the guyed tower under three types of sea states are of interest: (1) operating sea states, (2) 100-year design sea states, and (3) rare intense sea states.

<u>Operating Sea States</u> - The selection strength and stiffness of the mooring system are influenced by the operating sea states.

The operating condition usually includes some nominal environmental loadings. This condition may govern the design of a number of platform elements because of the lower allowable stresses. Furthermore, the smaller wave periods associated with the operating waves lead to higher amplifications in the flexural and torsional modes which in turn produce most of the stresses in the tower. The required axial capacity of the piles may also be controlled by the operating condition because of the higher factor of safety specified for this condition and due to the fact that the contribution of the design environmental loads to foundation forces is relatively small. The motions of the platform under operating conditions must be analyzed to ensure comfort of personnel.

Since the platform response is linear for this range of mooring system behavior linearized frequncy domain methods could be used for such applications.

Design condition refers to the behavior of the platform subjected to the 100 year storm. Dynamic analysis of the platform must be performed to determine member forces, deflections and foundation reactions. The overall design of the guyed tower is dictated by this condition. Satisfactory performance of the conductors is an important design consideration (1, 3).

A time history approach should be used in determining the response of tower. Fundamental to the time domain approach is the use of realistic wave force histories. Irregular representation of the sea state which contains proper distribution of energy associated with the various wave components that make up the sea must be used in such dynamic analysis. Since the dynamic response of structures is sensitive to the specific features of the exciting wave forces, a number of wave force histories of representative characteristics should be used in design.

A technique that combines the design wave approach used in conventional static analysis and the irregular representation of the sea state for dynamic analysis is often used in practice. In this approach the inertial loads due to platform motions are computed from the dynamic analysis using an irregular representation of the sea state. These loads are next combined with static wave forces based on nonlinear wave theories such as Stokes V or Stream Function.

Since the guyed tower is dynamically sensitive to a number of environmental effects the interaction of various parameters should be investigated. The effect of wave-current interaction is discussed in Reference 1 and a plot from there is shown in Figure 3-4. Dynamic sensitivity of guyed tower platforms to wind excitations is discussed in Reference 15.

Finally, performance of the guyed tower should be assessed for overload conditions. In the design of conventional platforms, the safety of the structure under extreme environmental conditions is ensured by the use of appropriate factors of safety in the strength and capacity of platform elements. Examples are the use of allowable stresses less than the failure stress of the material and the use of appropriate factors of safety in determining foundation capacity. Besides the use of factors of safety, the integrity of the guyed tower must be examined for specific overload conditions since the behavior of the guyed tower is nonlinear and explicit analyses are needed to identify the failure modes of a guyed tower. The overall failure could be due either to overstressing of the piles or to the P-Delta forces which are the overturning moments produced by gravity loads acting through the lateral deflections. Hence the preferred approach should be to limit the lateral deflections of the platform, limit the stresses in the piles and ensure that the mooring lines can accommodate the excursions of the tower without reaching their breaking strength. Stated differently, while the behavior of conventional fixed platforms is controlled by loads, the guyed tower behavior is governed by deflections.

Damaged condition analysis refers to the investigation of the safety of the guyed tower assuming failure of one or more critical elements of the system. Since the mooring system is vital to the integrity of the guyed tower, the performance of the guyed tower must be investigated assuming failure of one or more of the mooring lines. Deflections of the platform should not be excessive and the forces in the platform elements such as the cables, structural members and piles should be within allowable limits. The overall integrity of the platform with particular reference to its overturning tendency must also be examined.

Some results for a representative guyed tower in 1,500-ft. water depth are presented here for purposes of illustration. A perspective view of the tower is shown in Figure 3-5. The horizontal force vs.

3.5 FATIGUE ANALYSIS

Fatigue behavior of the tower and the guying system must be investigated. Since the fundamental natural period of the platform is much larger than the wave periods the associated vibrational modes do not contribute to the fatigue damage of the guyed tower. However, the flexural modes of the guyed tower play the same role as the fundamental period of a fixed platform as far as the fatigue behavior is concerned. In particular, as the flexural and torsional periods of the guyed tower approach the 4 to 6 seconds range, fatigue becomes a major design consideration for the guyed tower.

The cumulative fatigue damage of the guyed tower can be determined using the probabilistic spectral fatigue analysis procedures currently used for deepwater fixed platforms (16-19). This linearized frequency domain approach is valid for the guyed tower since the mooring system response fall into the linear ranges for most of the every day sea states which contribute to the fatigue damage of the tower.

3.5.1 Spectral Dynamic Fatigue Analysis

The theoretical background on the spectral dynamic fatigue analysis is described in References 16 and 17. Using a time domain dynamic structural analysis procedure, the transfer functions for hot-spot stresses at selected number of points around the ends of each member in the structure are generated as a function of wave frequncy and direction. For each stress location of interest, the response spectrum is computed by multiplying the transfer function by the wave spectrum applicable to that particular sea state. The response spectrum is then used to generate short term stress statistics applying a Rayleigh distribution for the stress ranges. The RMS response amplitude and mean zero-crossing period of the stress response are also computed. The short term stress statistics are obtained for each sea state in the long term wave data and fatigue damages are calculated using Miner's rule.

The following important aspects of detailed fatigue analysis are discussed in subsequnt sections.

1. Time Domain Structural Analysis
2. Stress Concentration Factors (SCF)
3. Fatigue (S-N) Curves
4. Computation of Cumulative Damage Ratio (CDR)

Time Domain Structural Analysis - The structural analysis must be performed on three dimensional space frame models of the platform consisting of all major structural framing. Appurtenances such as conductors, boatlandings, etc., which contribute to wave forces were adequately simulated. The first few frequencies and associated mode shapes must be computed.

The platform is subjected to each of the waves selected for generating transfer functions and dynamic response determined by integrating the modal equations of motion. Two percent modal damping is appropriate for each mode (6).

The time-step analysis is continued till a steady state condition is achieved. During the last cycle of response, the member end forces are computed for ten or more wave positions corresponding to time steps at equal increments in a complete wave cycle. The stress data at these wave positions are interpolated to obtain the stress cycle from which the stress range is determined.

The structural analysis described above which adequately represents the dynamic effects neglects the static contribution due to the higher modes. The importance of including the static effects has been recognized and an elegant approach called the static plus inertial can be used (18) to account for the static response of higher modes. The basis of this procedure is as follows. Let $X(t)$ be the vector representing joint displacements at time t. The displacements can be considered in two parts, static and inertial. That is

$$X(t) = X(t)_{static} + X(t)_{inertial}$$

The static displacements are obtained by the regular static analysis using a complete set of wave loads. The inertial displacements are computed from the dynamic analysis mentioned earlier. Note that inertial displacements do not include the contribution of static effects, it is the displacement due only to inertial and damping forces.

Fatigue (S-N) Curves - The S-N curves for tubular connection of platforms can be selected based on the criteria specified in API RP2A (6).

Stress Concentration Factors (SCF) - Empirical hot spot stress concentration factors are developed for the ends of each member in the structure. These factors recognize separately the effects of axial load, in-plane bending and out-of-plane bending in a branch member at a tubular connection. The various types of formulae used to compute stress concentration factors are available in References 19-21.

Computation of Cumulative Fatigue Damage Ratio - The wave spectrum $S_\nu(\omega)$ can be related to the response spectrum $S_\sigma(\omega)$ by the transfer function $T_\sigma(\omega)$ as follows:

$$S_\sigma(\omega) = T_\sigma(\omega) \cdot S_\nu(\omega)$$

In this case S_σ is the hot-spot stress at selected circumferential points of a specified member end. The transfer function, T_σ, is defined for this section as the square of the ratio of the response amplitude to the wave amplitude. The stress spectrum is obtained using the above procedure for all combinations of sea states and incident directions for selected joints and member ends in the jacket.

Let m_i be the ith spectral moment associated with the stress spectrum and defined by

$$m_i = \int_0^\infty \omega^i S_\sigma(\omega)\, d\omega$$

with i = 0, 1, 2, . . . etc.

The probability distribution of the stress ranges can be approximated by a Rayleigh distribution of the form:

$$P(r) = \frac{r}{4m_o} e^{-r^2/8m_o}$$

The average period of the stress response cycle is computed from

$$\zeta_{av} = 2\pi \left\{\frac{m_o}{m_2}\right\}^{1/2}$$

The fatigue damage is computed using the well-known Miner's rule. For a continuous stationary random stress process (15), CDR is given by,

$$CDR = \frac{T}{\zeta_{av}} \int_0^\infty \frac{P(r)}{N(r)}\, dr$$

where:

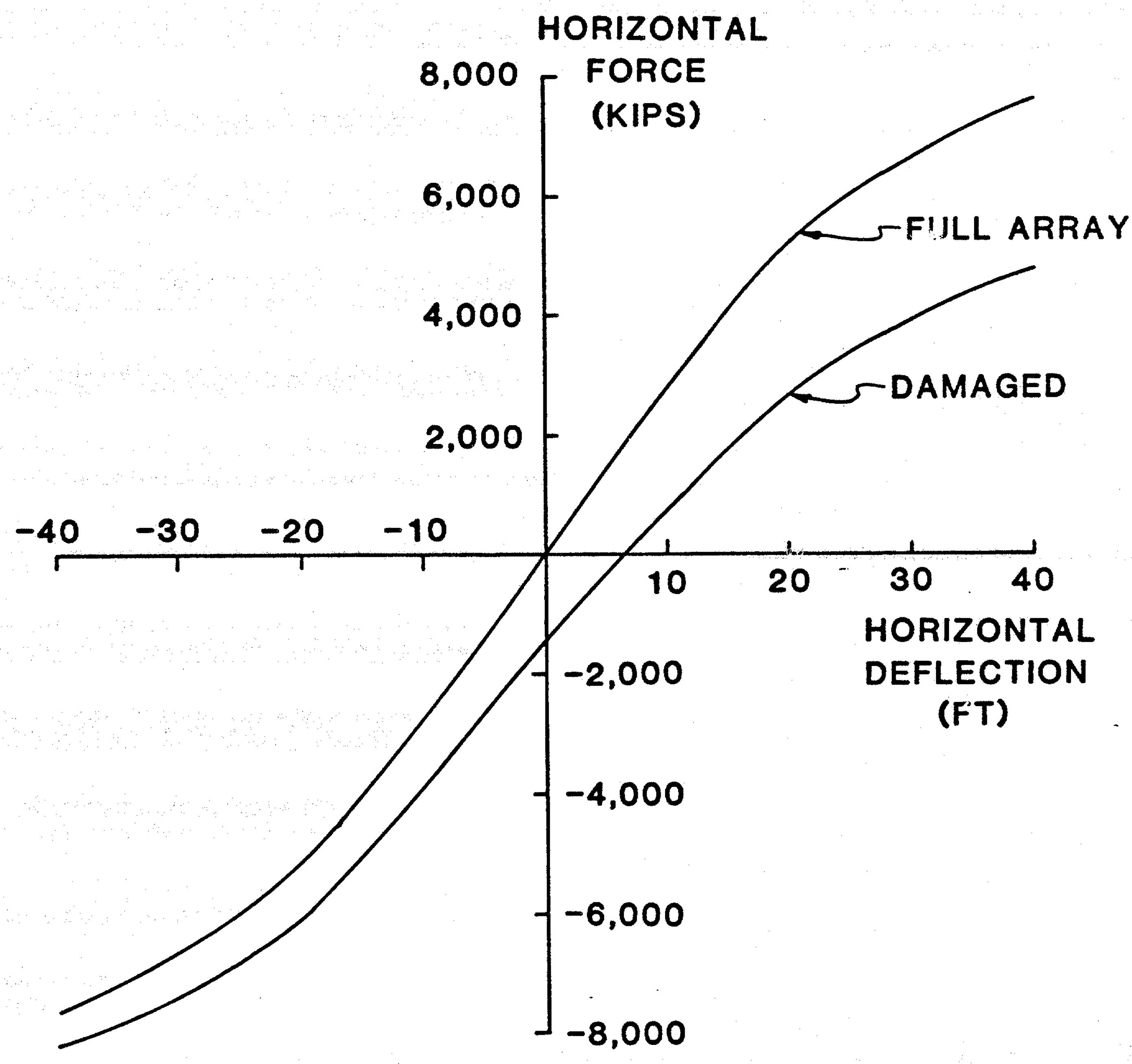
T	=	Time duration of the random stress process.
ζ_{av}	=	Average period for stress variation in the random process.
$P(r)$	=	Probability density function of stress range.
$N(r)$	=	Average number of cycles to failure at a stress range r.

Once P(r) and N(r) are defined, the integration in the CDR equation is carried out numerically.

The fatigue life of the guylines should also be investigated considering the axial and bending behavior. A procedure similar to that described above can be used to generate the transfer functions for stress ranges. Fatigue characteristics of steel rope and strand are available in references 22 and 23. Portions of the guylines that pass through paraleads and bending shoes are particularly susceptible to fatigue failure and should be investigated

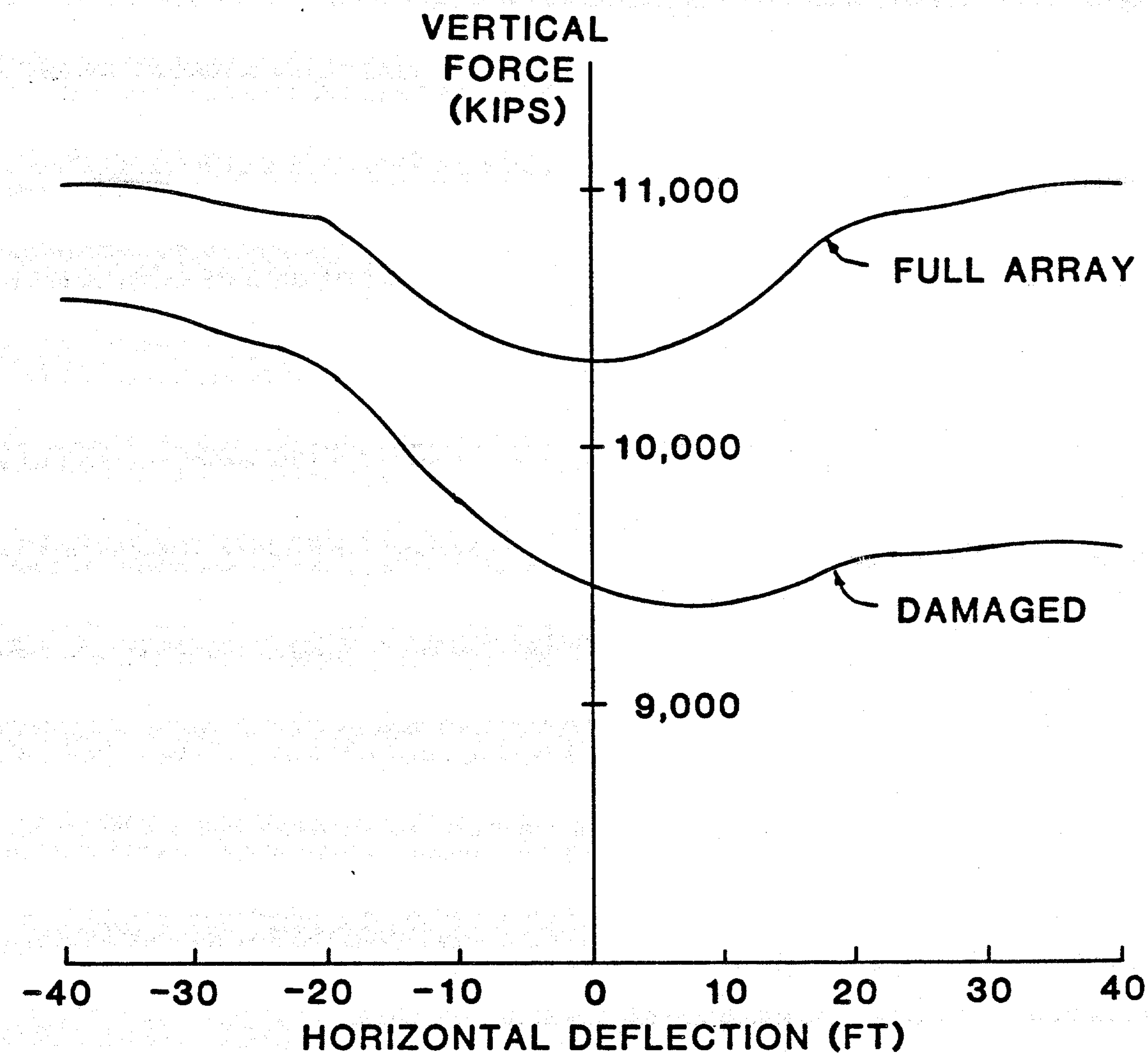
3.6 HUMAN COMFORT TO DECK MOVEMENT

Excessive platform motion can cause operational problems and affect personnel efficiency and should be investigated. The acceleration at deck levels should be computed for various operational sea states and then compared with tolerance levels, specified in various publications, e.g., Ref. 24.



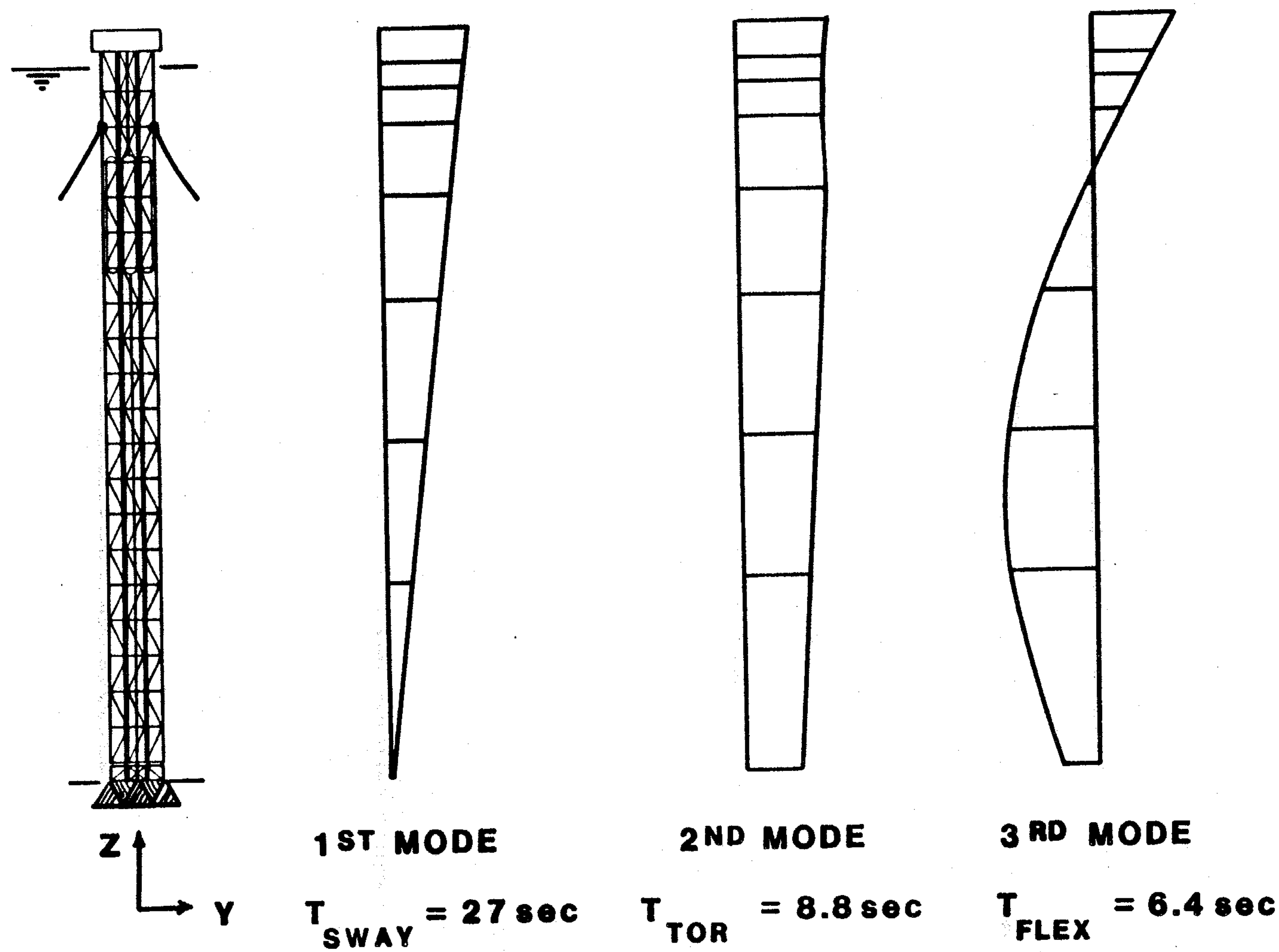
HORIZONTAL FORCE VS. HORIZONTAL DEFLECTION

FIGURE 3-1



VERTICAL FORCE VS. HORIZONTAL DEFLECTION

FIGURE 3-2



COUPLED MODE SHAPES IN THE Y-DIRECTION AND ROTATION

FIGURE 3-3

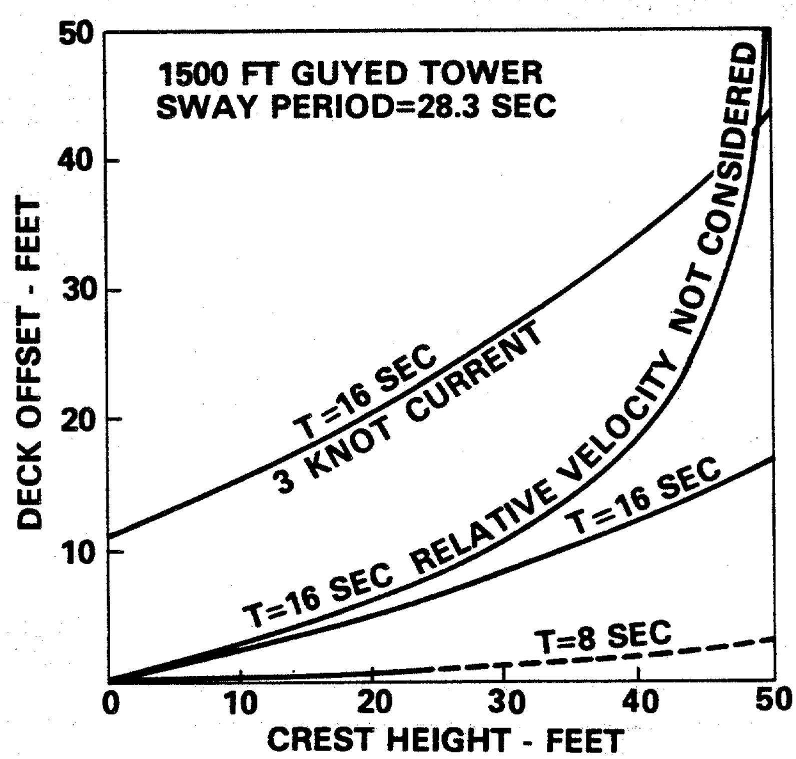


FIGURE 3-4 (REF. 1)

WAVE-CURRENT INTERACTION

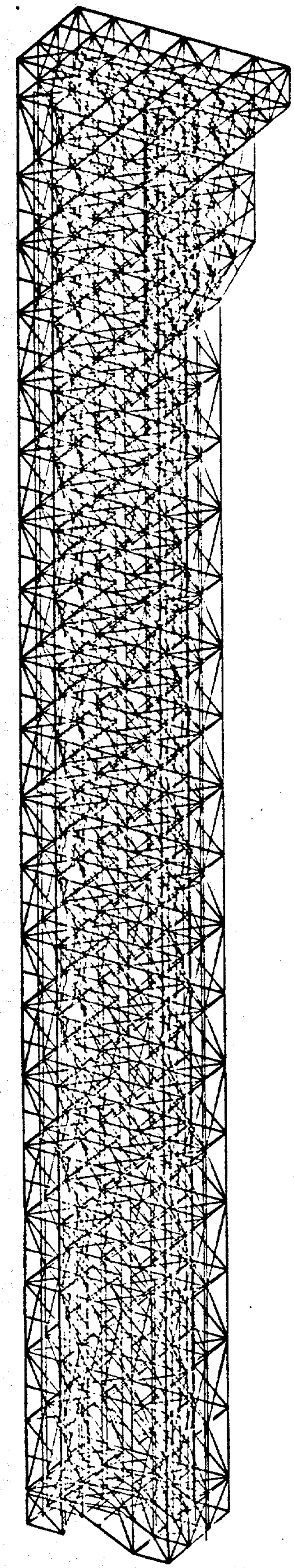
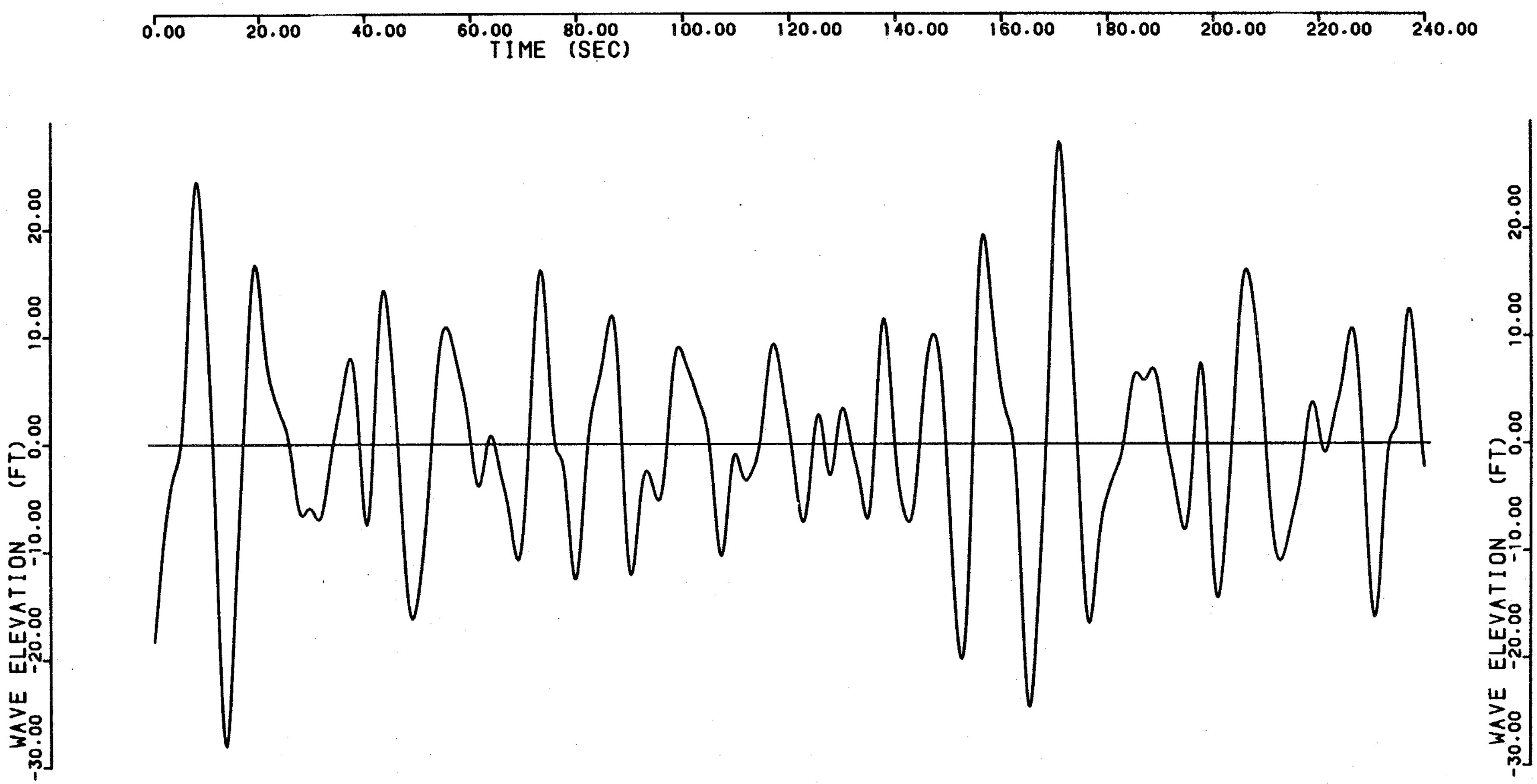
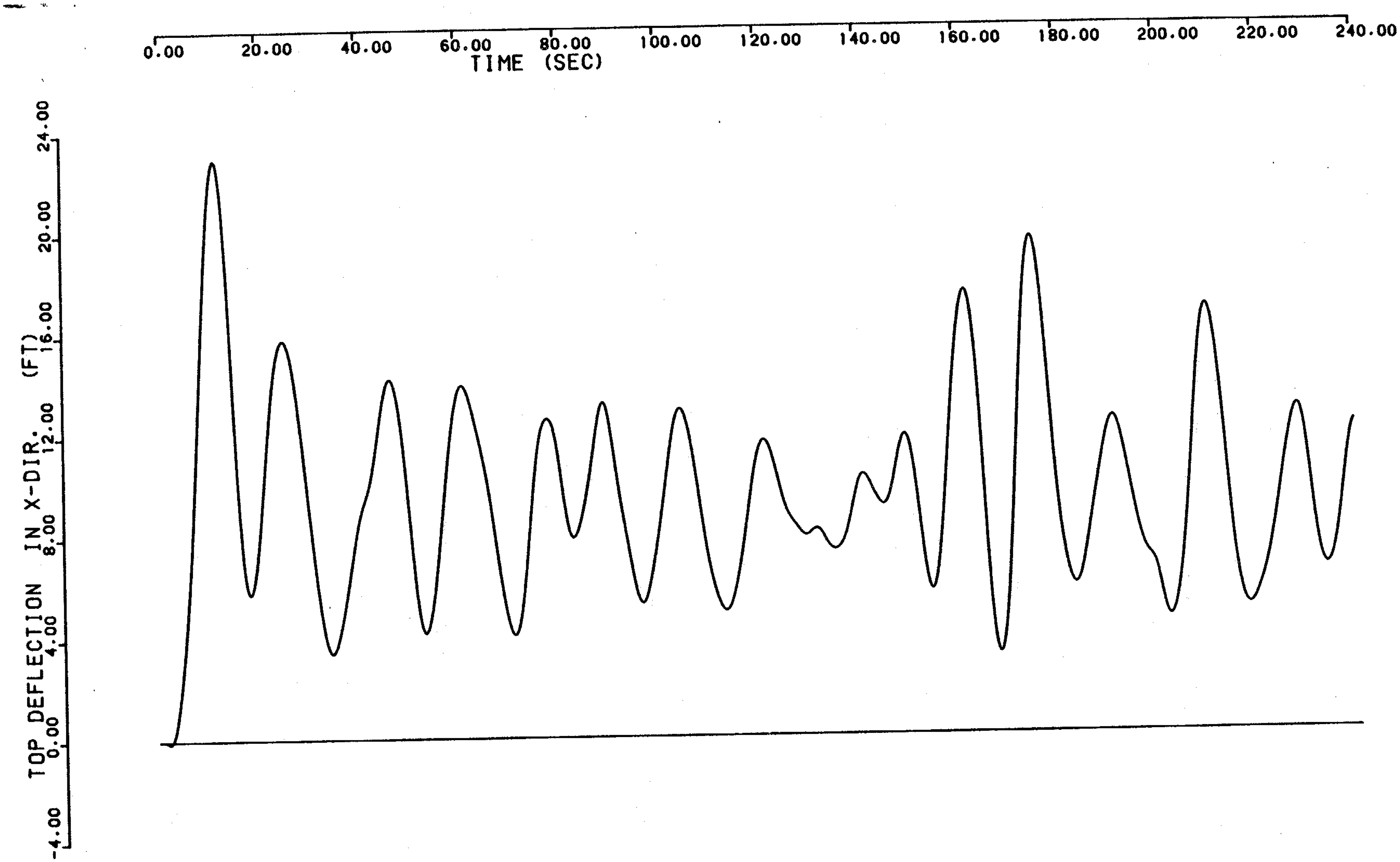


FIGURE 3-5
GUYED TOWER - PERSPECTIVE



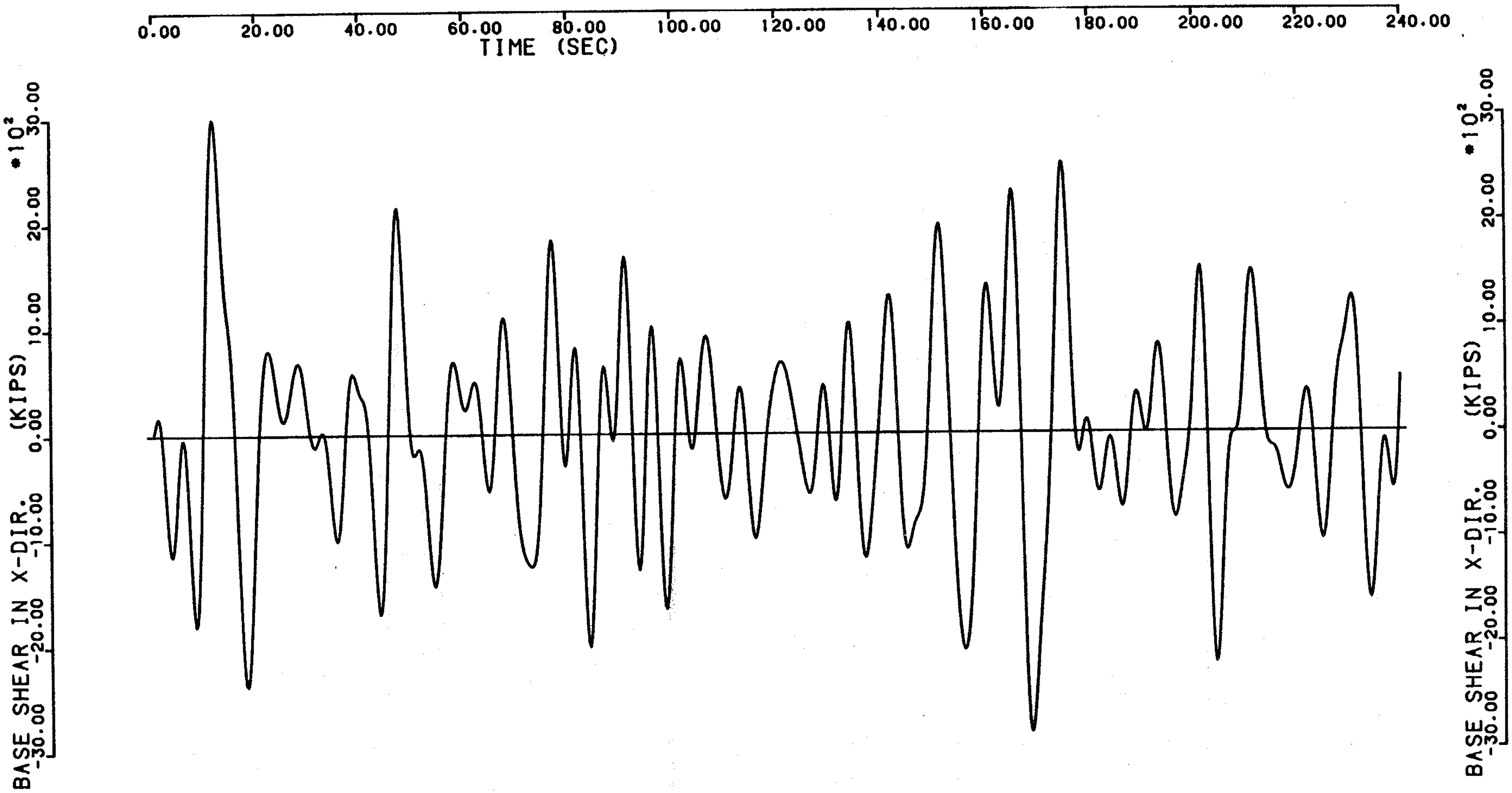
WAVE ELEVATION VS. TIME

FIGURE 3-6



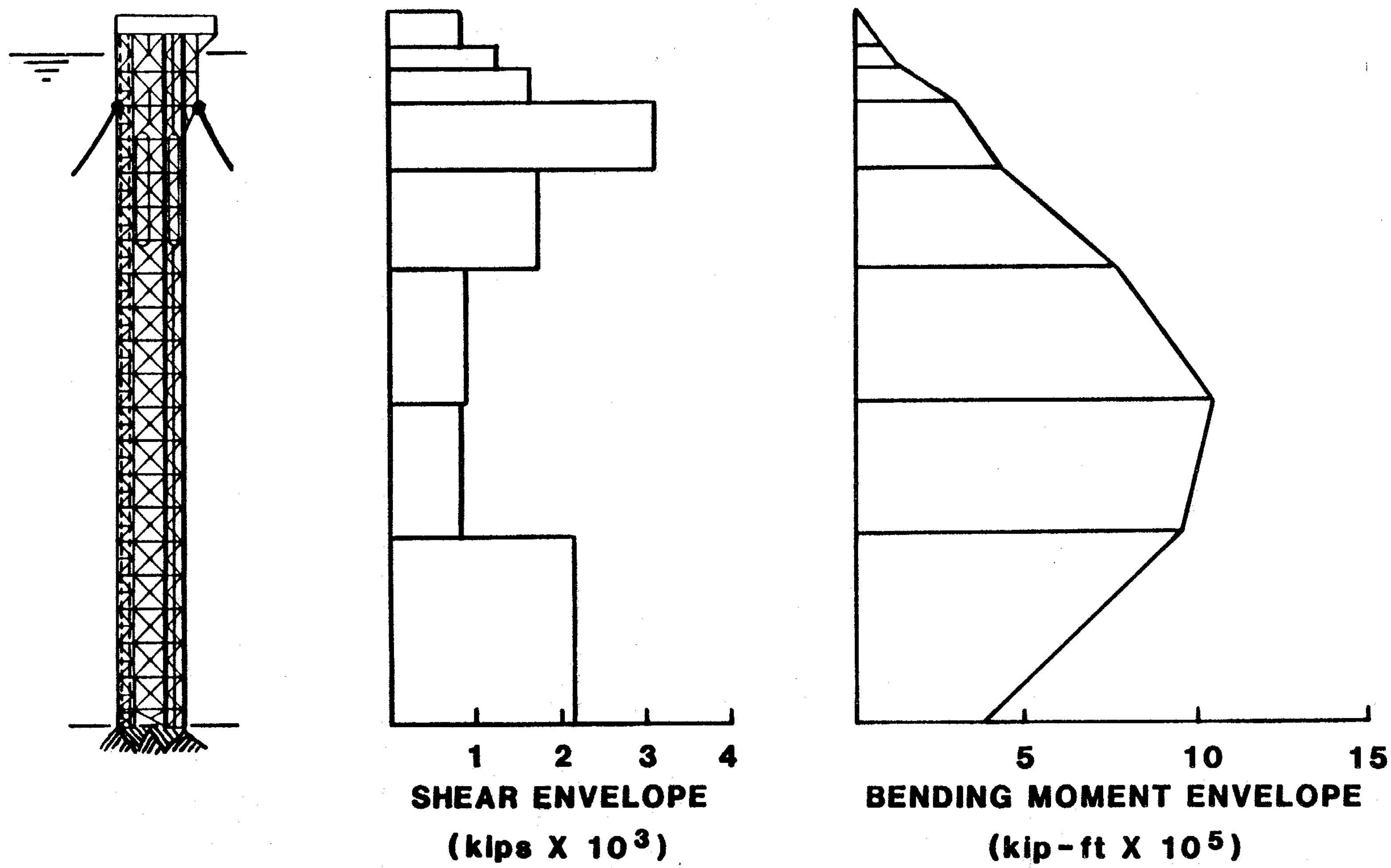
DECK OFFSET VS. TIME

FIGURE 3-7



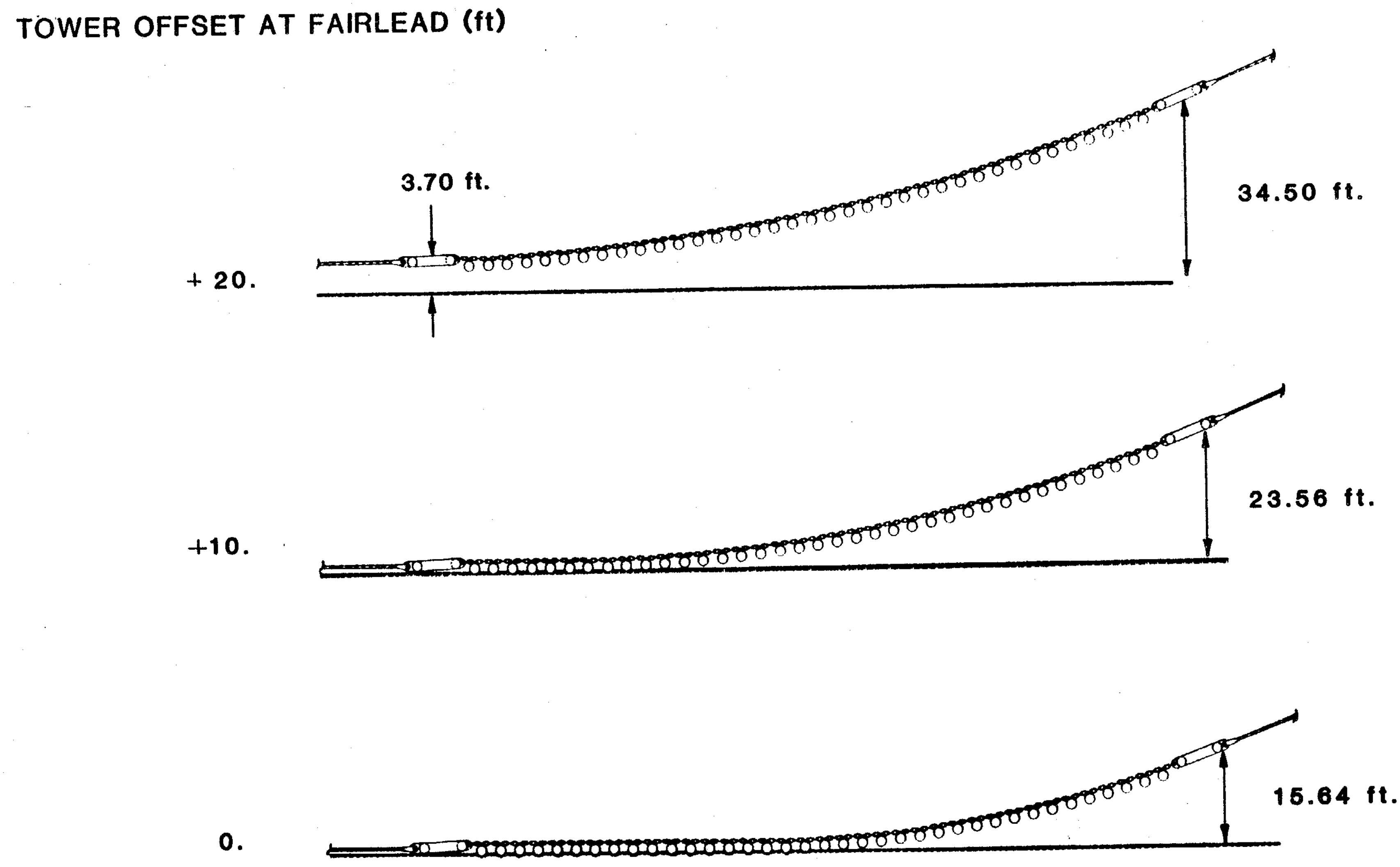
BASE SHEAR VS. TIME

FIGURE 3-8



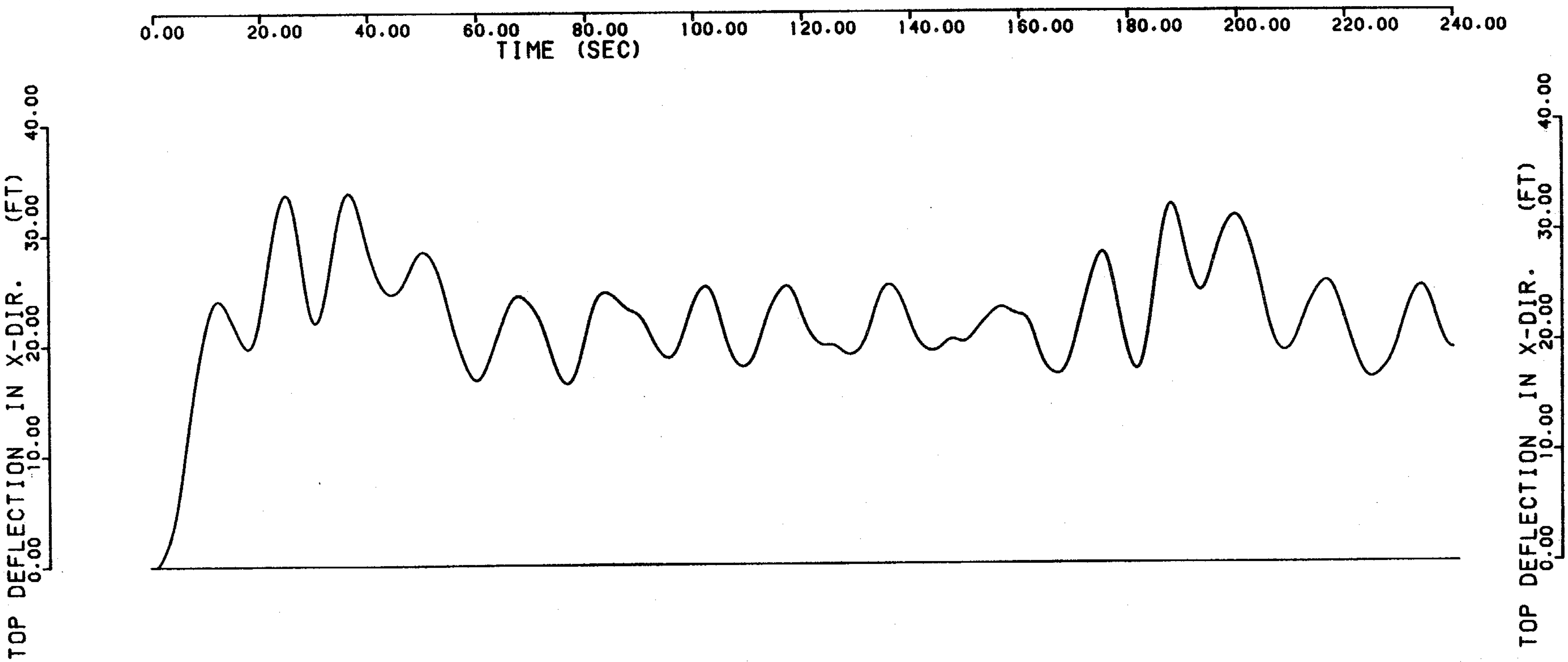
DYNAMIC RESPONSE ENVELOPES

FIGURE 3-9



CLUMPWEIGHT PROFILES

FIGURE 3-10



EFFECT OF LOSING TWO IN LINE TAUT MOORING LINES

FIGURE 3-11

4. METHODS OF GUYED TOWER ANALYSIS

Two different approaches can be effectively used to predict the response of guyed tower platforms subjected to environmental forces. The first approach called coupled analysis incorporates an explicit model of the mooring system to predict the response of the tower and the mooring system in a single analysis. In the second approach called uncoupled analysis two separate analyses are performed; first to predict the overall platform response and the second to predict the response of the mooring system itself. Obviously, the first technique is more accurate and much more computationally involved. If judiciously used the second approach can also yield reliable results for design applications.

4.1 COUPLED ANALYSIS

The analytical model consists of the tower, mooring lines and the foundation system (Fig. 4-1). The tower could be modelled as a three dimensional space frame idealized with elastic beam elements. A simplified model of the tower which properly reflects the elastic stiffness and hydrodynamic properties could also be used. The foundation system can be modelled in an equivalent linear manner or can be modelled in detail using elastic beam elements to represent piles and nonlinear p-y and t-z curves to account for nonlinear hysteretic behavior of the soil. The specific details of the model will depend upon the features of the computer program being used. If a linearized foundation is used in the overall analysis, a separate analysis of the foundation which accounts for the nonlinear soil behavior should be performed to determine local stresses in the piles.

The mooring system should be modelled accurately incorporating the lead lines, clumpweights, and anchor lines. The suction forces exerted on the clumpweight by the seafloor soil should also be included. The cables should be modelled by finite elements which account for their large deflection behavior.

The governing equations of motion can be written as

$$M\ddot{x} + C\dot{x} + Kx = F \tag{1}$$

in which M is the mass matrix and should be computed as discussed in Section 2 including the contribution of hydrodynamic mass. The damping matrix C should include structural and foundation damping and the contribution of hydrodynamic radiation damping. The stiffness matrix K includes contribution of the tower, foundation and mooring lines and is nonlinear.

The large deflection effects of the guyed tower, that is the additional overturning effect produced by the vertical loads acting through tower deflections should be properly included in the analysis. This can be done by including the geometric stiffness in the formulation of element stiffness matrix or can be accounted for in a simplified manner. The latter will result in additional lateral forces being applied to the platform.

The right hand side of Eq. 1 should include both permanent and environmental forces. The former includes dead and live loads, hydrostatic forces and pretension in the mooring lines. The environmental loads are wind, wave, current and earthquake induced ground motions, if applicable.

The equation of motion can be numerically integrated in time domain and various response parameters such as deflections, base shear, member forces, etc. can be determined.

Obviously the coupled analysis is extremely complex and can only be justified in final design stages as a verification tool.

4.2 UNCOUPLED ANALYSIS

The fundamental assumption made in this case is that the dynamic behavior of the mooring sytem does not substantially affect the overall response of the guyed tower. The tower response is computed using a simplified model of the mooring system and subseqently a separate analysis of the mooring system is performed to determine the individual guyline forces.

In the uncoupled analysis of the guyed tower platform, the mooring system is modeled as a massless spring having nonlinear force-displacement behavior derived from a static or dynamic analysis of the mooring system. The lateral force vs horizontal deflection at the point of attachement of the mooring system to the tower is shown plotted in Figure 3-5. The corrresponding plot of the horizontal displacement vs vertical force is shown in Figure 3-6. The nonzero ordinate in the latter plot represents the vertical force introduced on the platform due to pretension in the cables.

Having determined the overall platform response, a separate analysis of the mooring system can be performed. The forces or displacements at the tower-mooring system interface can be used as input in this analysis which could include the dynamic response of the guylines.

The uncoupled analysis of the mooring system can be based on any one of the following methods.

1. Direct Integration Method
2. Modified Modal Technique

The second approach is particularly suited for the uncoupled analysis because of its computational efficiency. The basis of this procedure is discussed in References 25-27 and is briefly reviewed here.

The equation of motion is written as

$$M\ddot{x} + C\dot{x} + Q = F(t) \tag{1}$$

in which

$$Q = (K_T + K_C)X \tag{2}$$

K_T = stiffness of the tower
K_C = stiffness of the cable

The cable stiffness can be written as,

$$K_C = K_{CI} - (K_{CI} - K_C) \tag{3}$$

Now, Eqn. (1) can be written as,

$$M\ddot{x} + C\dot{x} + K^{*}x = F(t) + F_N(t) \tag{4}$$

in which

$$K^{*} = K_T + K_{CI} \tag{5}$$
$$F_N(t) = (K_{CI} - K_C)X \tag{6}$$

Note that K_{CI} is the initial stiffness of the mooring system.

Eq. 4 can be uncoupled using the modal coordinates of the undamped system defined by

$$M\ddot{x} + K^{*}x = 0 \tag{7}$$

The excitation term on the right hand side includes the environmental forces as well as a correction term to account for the nonlinearities of the mooring system. Since the correction term is a function of the displacement, an iterative procedure is needed. The iteration can start by assuming that the displacement at any instant t is the same as that at a previous step and this scheme should converge in as few as two iterations to the correct answer.

Forces

The following forces should be considered in the guyed tower analysis.

- o Gravity loads
- o Buoyancy
- o Vertical component of the mooring system reaction
- o Wind
- o Wave
- o Current
- o Other environmental forces such as earthquake induced ground motions where applicable.

Computation of environmental forces follow procedures similar to that used for fixed platforms. Wind can be considered to act statically in most cases, but the dynamic effects of wind should be evaluated and included if found significant.

The wave force computation should follow guidelines discussed in Ref. 5. In particular the interaction of the wave and current and the relative velocity effects should be considered in the analysis. The force exerted by waves on a cylindrical member is computed from

$$F = C_D \frac{\rho}{2} D(u - \dot{x}) \left| u - \dot{x} \right| + C_M \frac{\pi}{4} D^2 \frac{du}{dt}$$

in which

F = fluid force per unit length acting normal to the axis of a member.

C_D = Drag coefficient

C_M = Mass coefficient

= Mass density of water

D = Diameter of member

u = Component of fluid velocity normal to the axis of the member

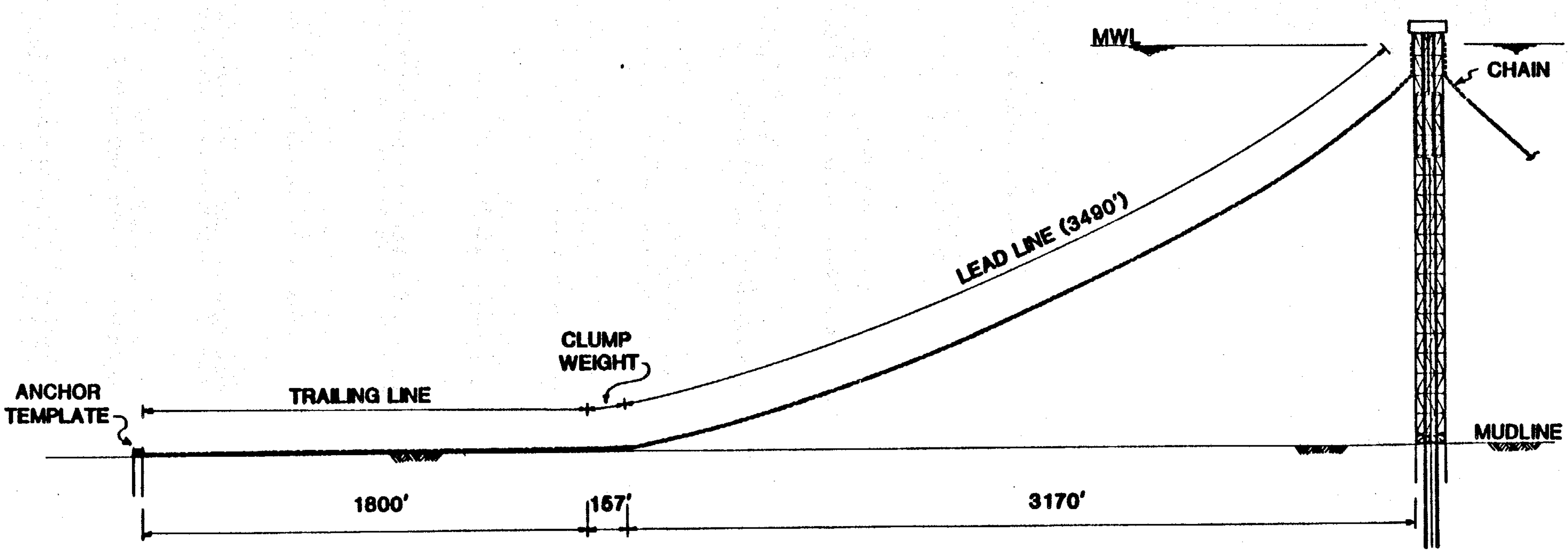
$\frac{du}{dt}$ = Component of fluid acceleration normal to the axis of the member

Current velocity should be added vectorially to the wave particle velocity and total should be interpreted as u in the above expression.

Fluid forces associated with the platform acceleration are accounted for by added mass.

<u>Wind</u> - The simultaneous action of wind should be considered together with wave and current forces. For most guyed tower configurations wind may be assumed to act in a static manner, at least in early design stages. Wind dynamics should be investigated if found appropriate (15).

References 28 and 29 present information on several aspects of guyed tower behavior.



GUYED TOWER MOORING SYSTEM WITH CLUMPWEIGHTS

FIGURE 4-1

5. FABRICATION AND INSTALLATION

The specific details of the guyed tower fabrication and installation procedure will depend on the nature of the particular project and also the availibility of suitable equipment. Only general conceptual procedures are discussed here. The discussion is intended to point out the unique aspects of guyed tower fabrication and installation operations.

Fabrication and installation of the deck can follow procedures used for fixed platforms and will not be discussed here.

5.1 FABRICATION PROCEDURES

Fabrication techniques closely resembling those for conventional fixed platforms can be used for the guyed tower. However, variations of the conventional procedures are needed to account for the particular features of the guyed tower. Specifically since the guyed tower is a slender tower the structural framing is relatively light, especially in the lower positions of the tower. With conventional bent roll-up erection procedures, it is possible that overstressing can occur. For this reason, the erection scheme shown in Figures 5-1 through 5-7 have been developed for the Lena guyed tower presently under fabrication. The intent is: (1) to fabricate substructures called "box sections" that have inherent structural strength without the addition of temporary construction bracing; and (2) to rotate these substructures into final position within the overall tower structures. A further advantage is that more work is done close to the ground, reducing the amount of expense associated with work high in the air.

The load-out and tie-down schemes will also be similar to those used for conventional platforms. The details will depend on the yard and the project particulars.

5.2 INSTALLATION PROCEDURES

Installation phase of a guyed tower involves following tasks:

1. Transportation of the tower from the fabrication yard to the installation site.

2. Launch, uprighting, and setting the tower on location.

3. Anchor pile, clump weight and guyline installation.

4. Pile installation

5. Deck installation.

Transportation - Guyed towers of the order of 1,000 ft. long can be transported as a single piece using existing barges (Figure 5-8). For towers longer than 1,000 ft. either new generation barges should be built or multipiece installation techniques similar to those used for deepwater jackets could be adapted. The horizontal mating of two guyed tower sections separately transported and launched is conceptually feasible.

Launch, Uprighting and Setting - Guyed towers can be launched either endways or sideways following conventional offshore procedures. A single piece side launch is shown in Figure 5.9. A two piece installation is shown in Figures 5-10 through 5-16. Uprighting and setting can also be performed by extending techniques used for conventional platforms. Specifically the tower will be upended by controlled flooding, brought alongside a derrick barge on a preset mooring and set on bottom (Figure 5-17).

Anchor Pile, Clump Weight and Guyline Installation - The installation of these components is unique to a guyed tower. A dynamically positioned construction vessel is preferred for the above operations. Detailed engineering is required for the planning of the above operations.

<u>Pile Installation</u> - Installation of the main and torsion piles can be accomplished using conventional techniques. In particular above-water hammer and follower arrangement can be used to drive the main and torsion piles.

<u>Deck Installation</u> - Modular deck installation can be accomplished by a derrick barge.

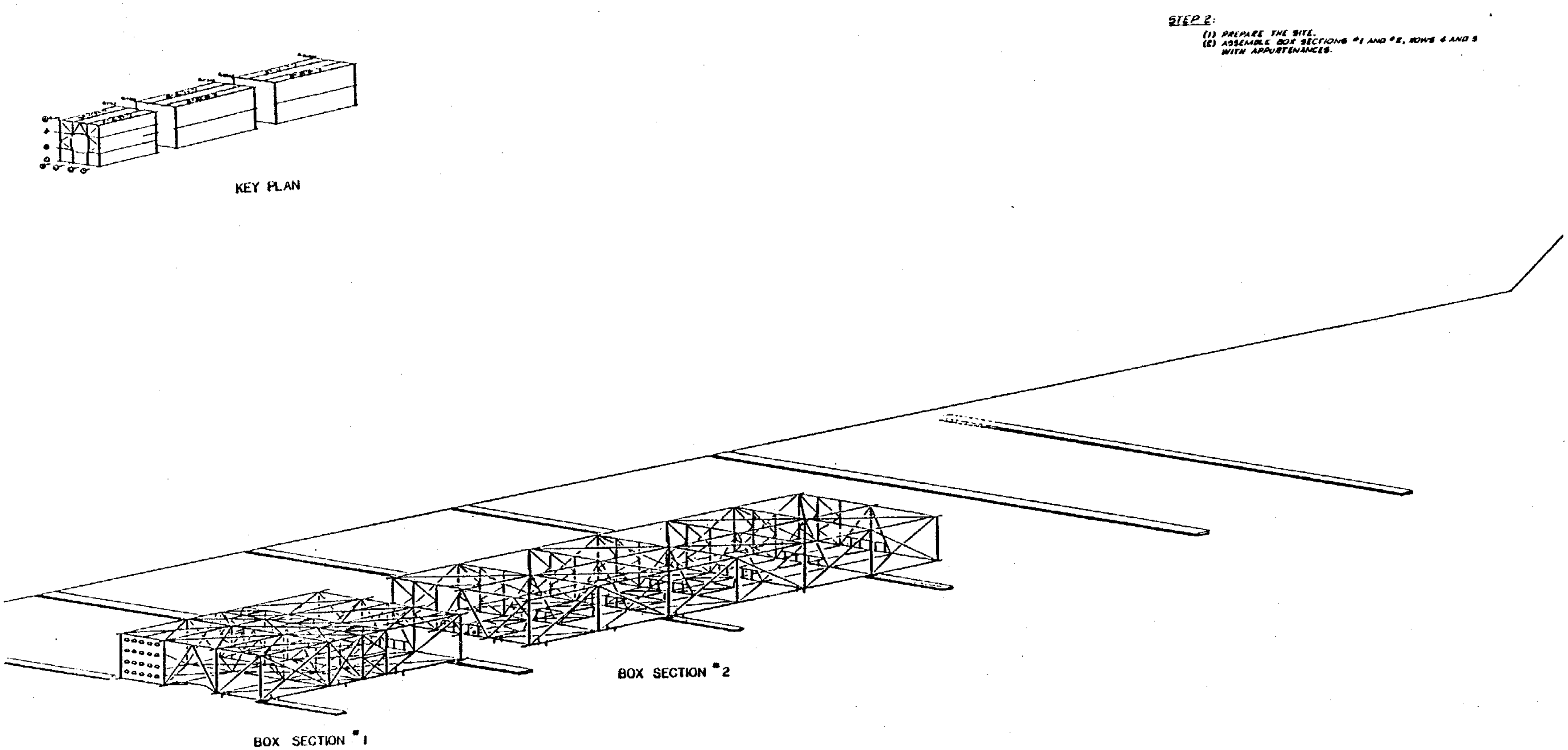


FIGURE 5-1
FABRICATION SEQUENCE

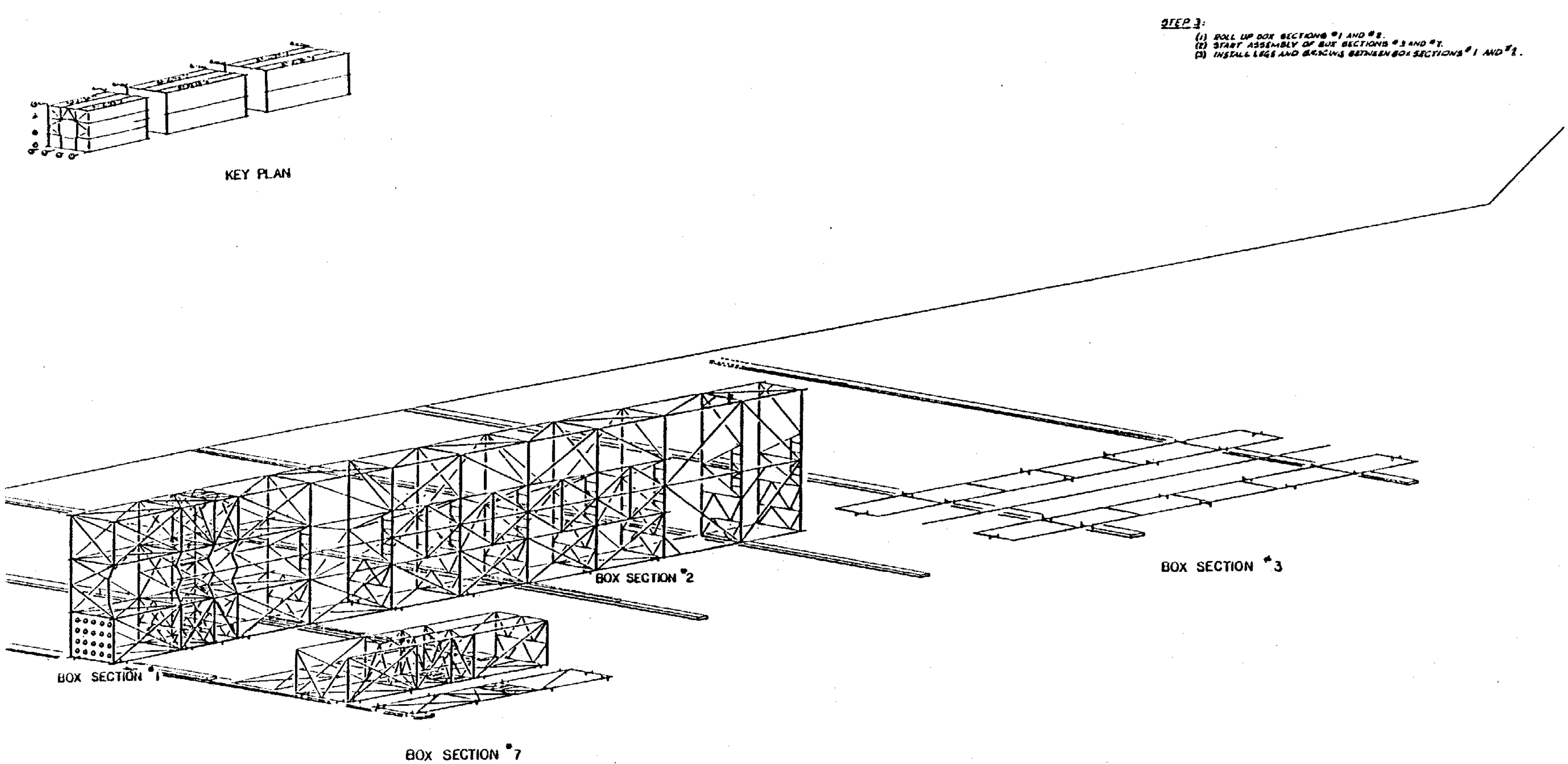


FIGURE 5-2
FABRICATION SEQUENCE

STEP 4:

(1) ERECT ELEVATION BRACING BETWEEN ROWS 2 AND 4 DELETING BRACING AT ROWS A, B, & C IN BUOYANCY TANK AREA.

(2) CONTINUE ASSEMBLY OF BOX SECTIONS #3 AND #7.

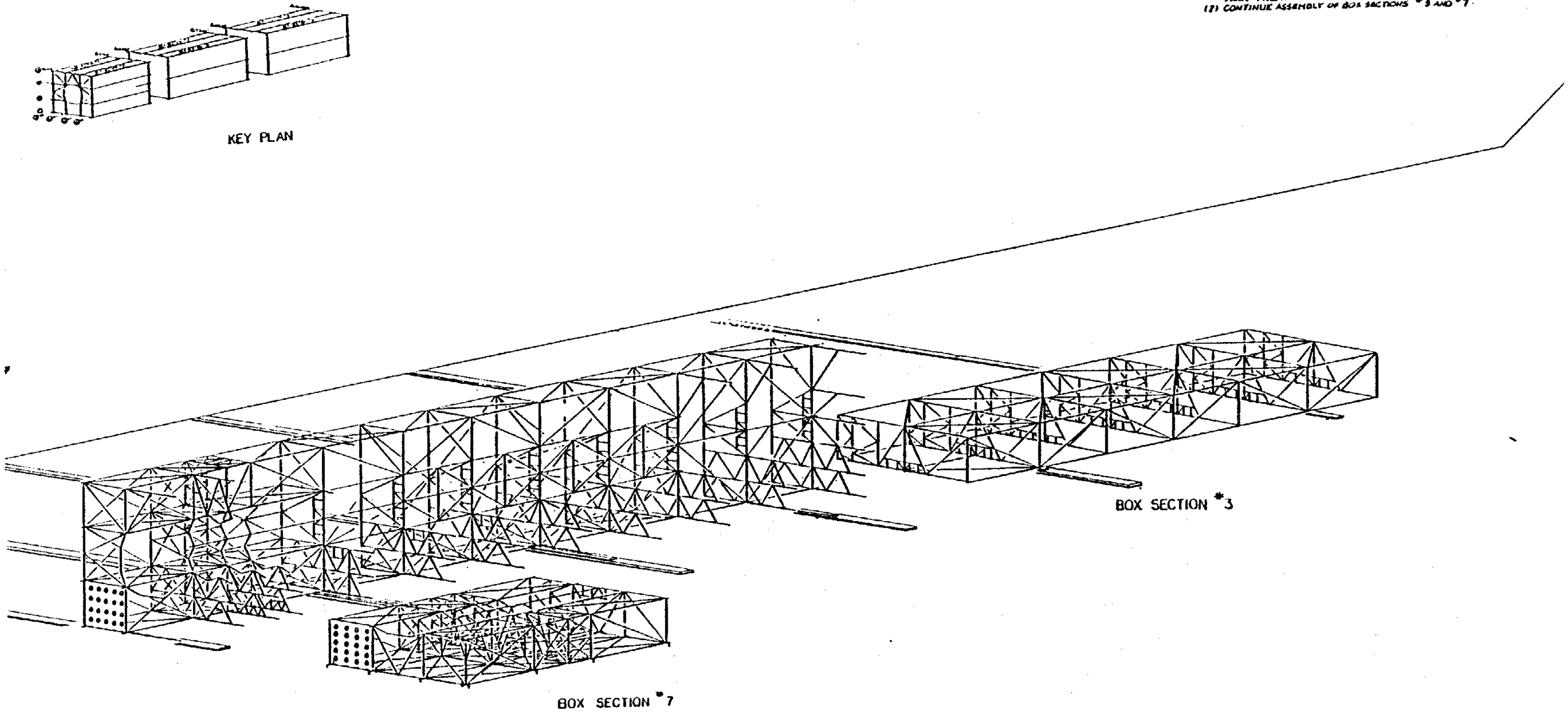


FIGURE 5-3

FABRICATION SEQUENCE

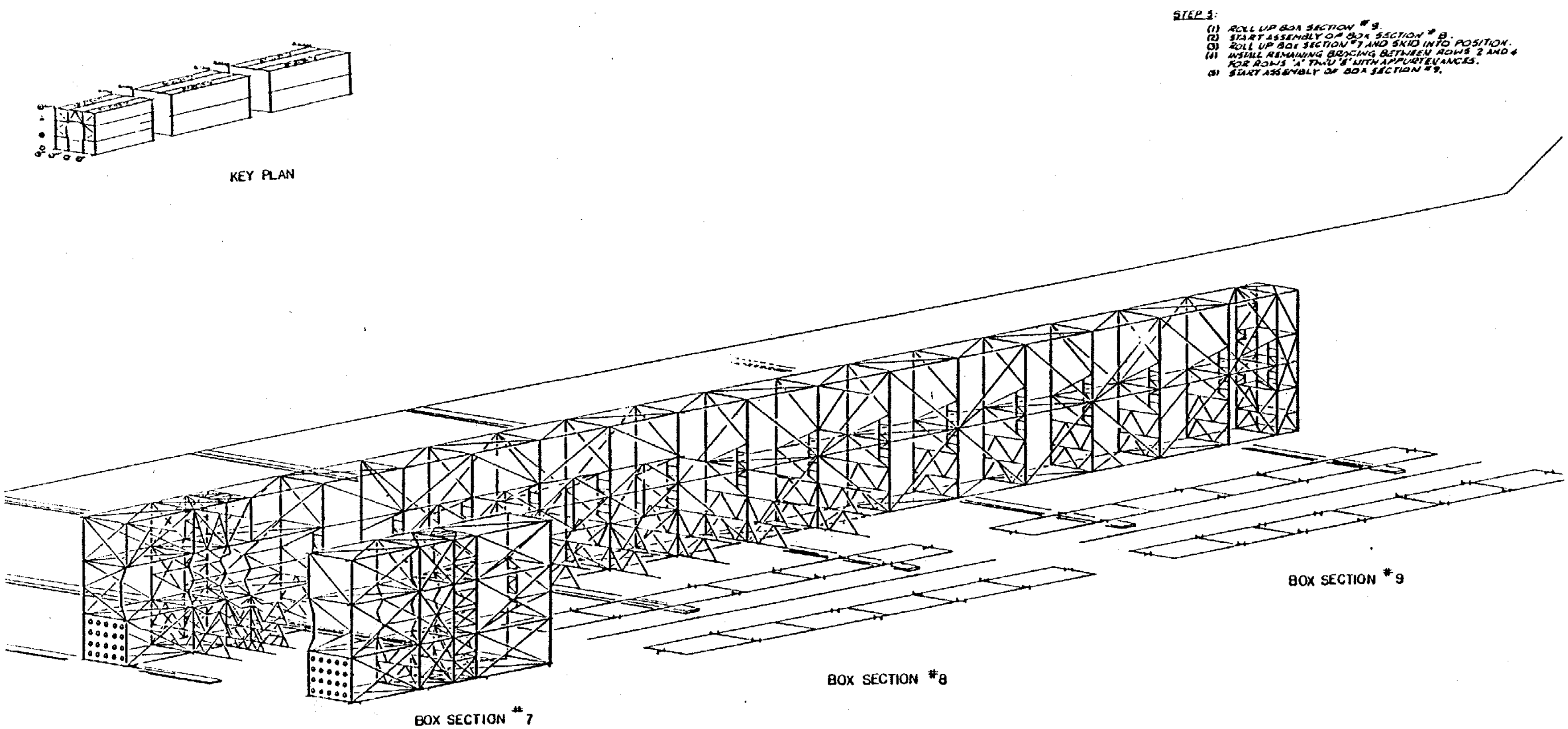
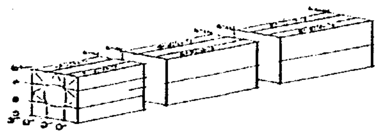


FIGURE 5-4
FABRICATION SEQUENCE



KEY PLAN

STEP 6:

(1) INSTALL LEGS AND BRACING BETWEEN BOX SECTION #2 AND #3.
(2) ERECT ELEVATION BRACING BETWEEN ROWS 2 AND 4 FOR ROWS 'A' THRU 'E' WITH APPURTENANCES.
(3) CONTINUE ASSEMBLY OF BOX SECTIONS #8 AND #9.

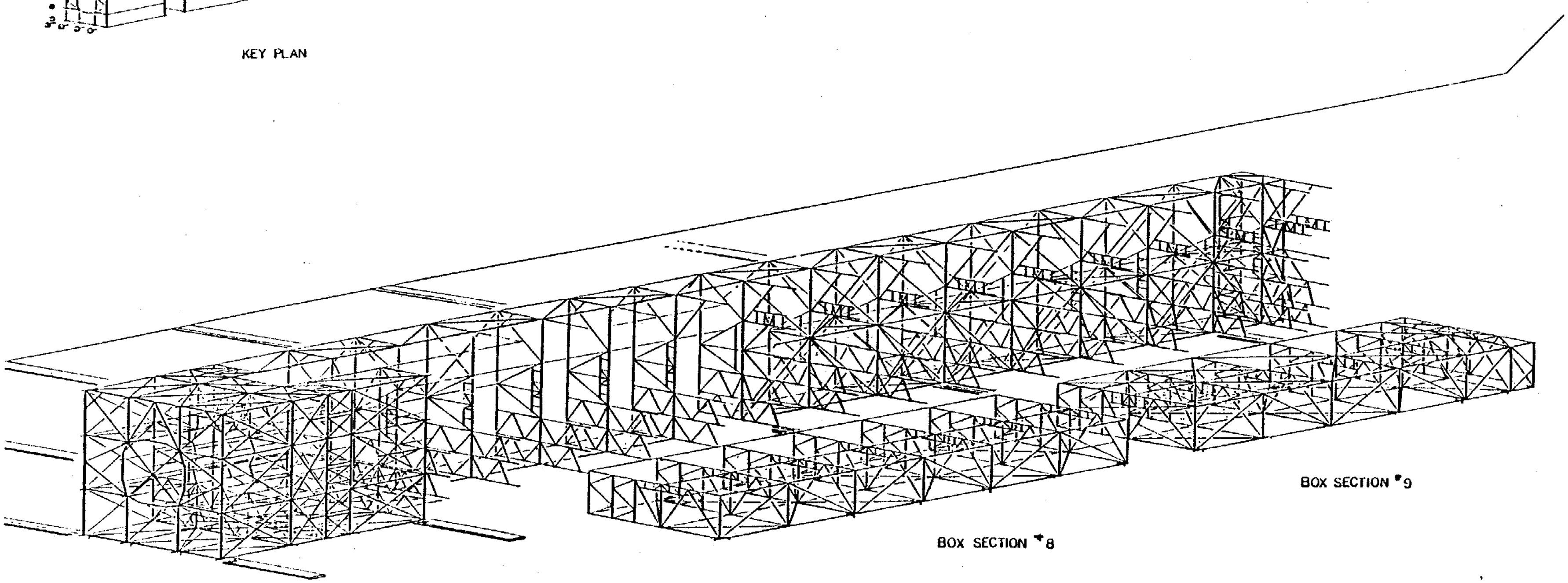


FIGURE 5-5
FABRICATION SEQUENCE

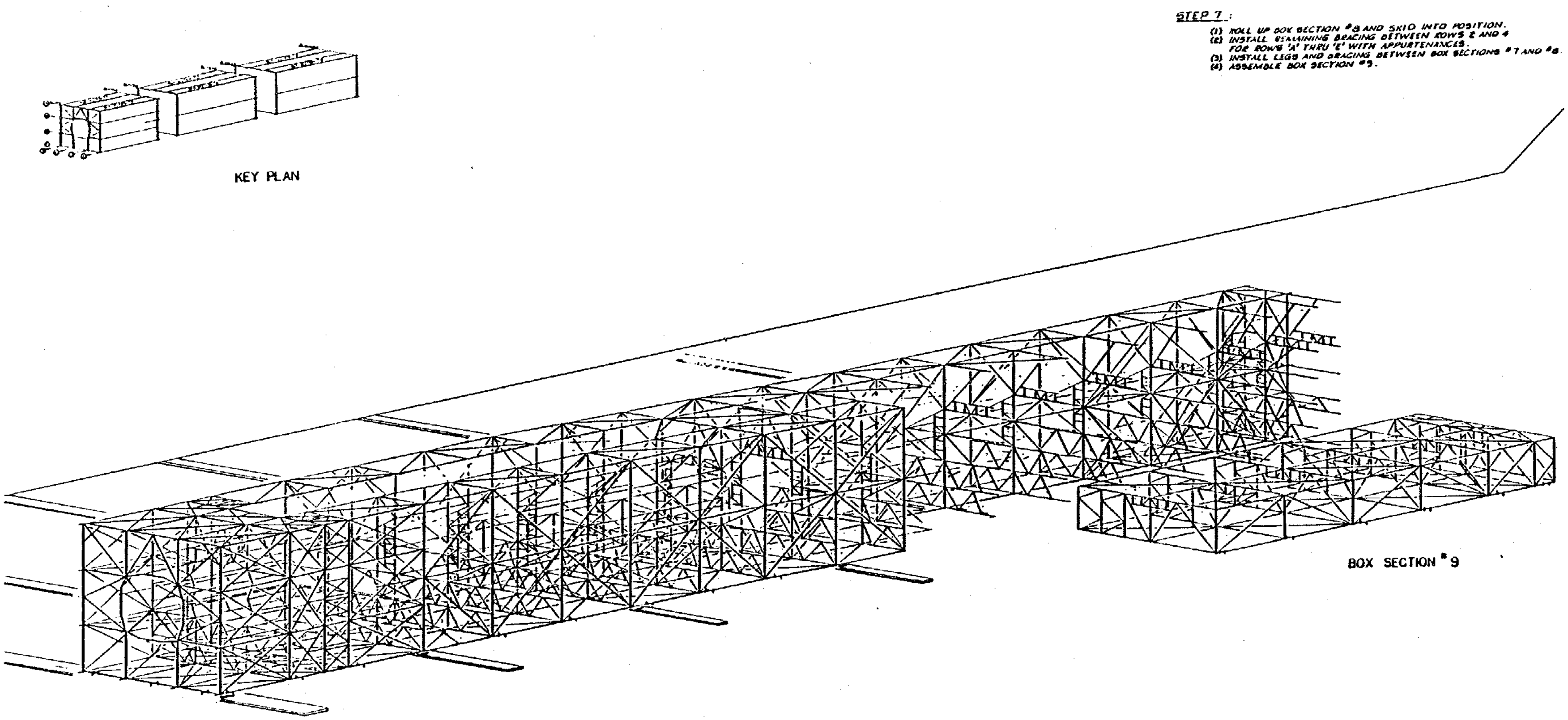


FIGURE 5-6
FABRICATION SEQUENCE

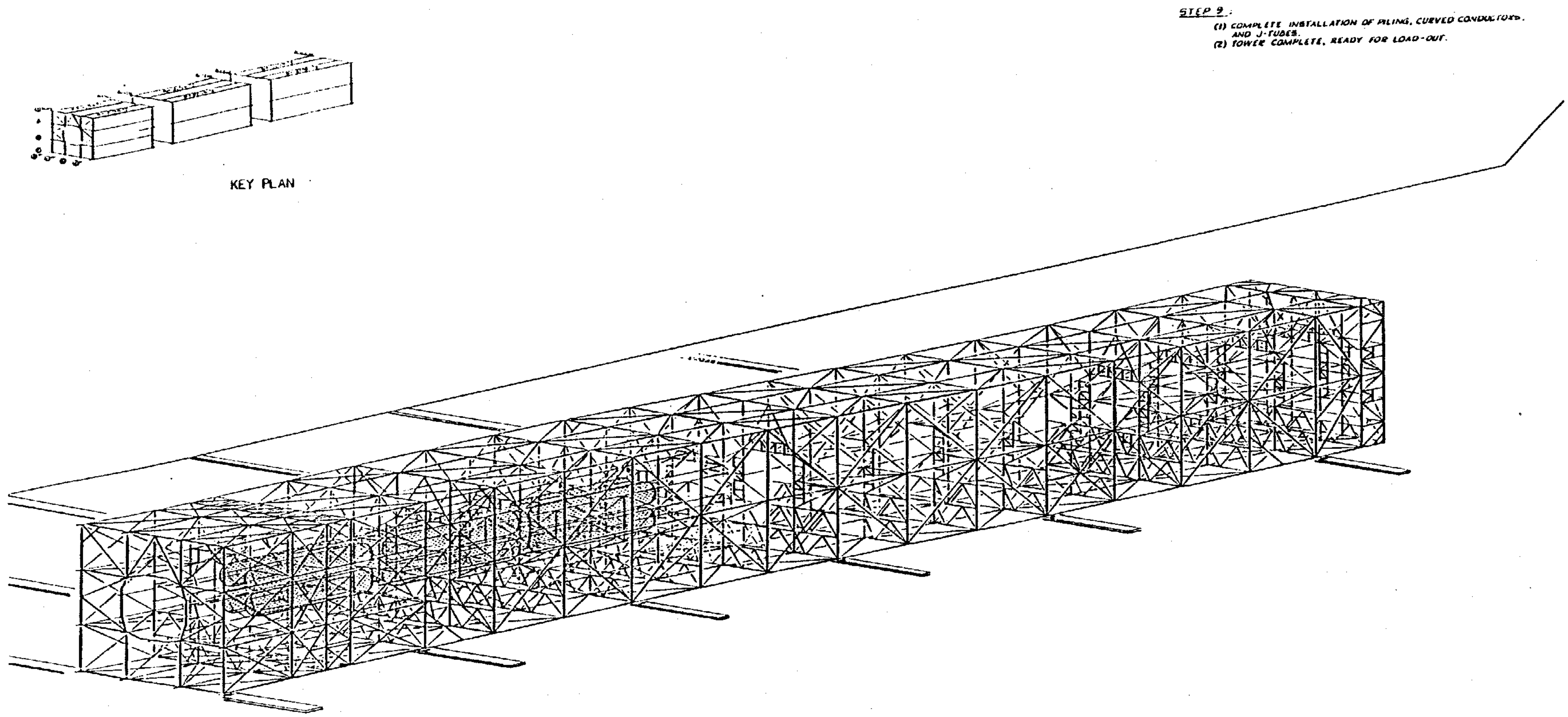


FIGURE 5-7
FABRICATION SEQUENCE

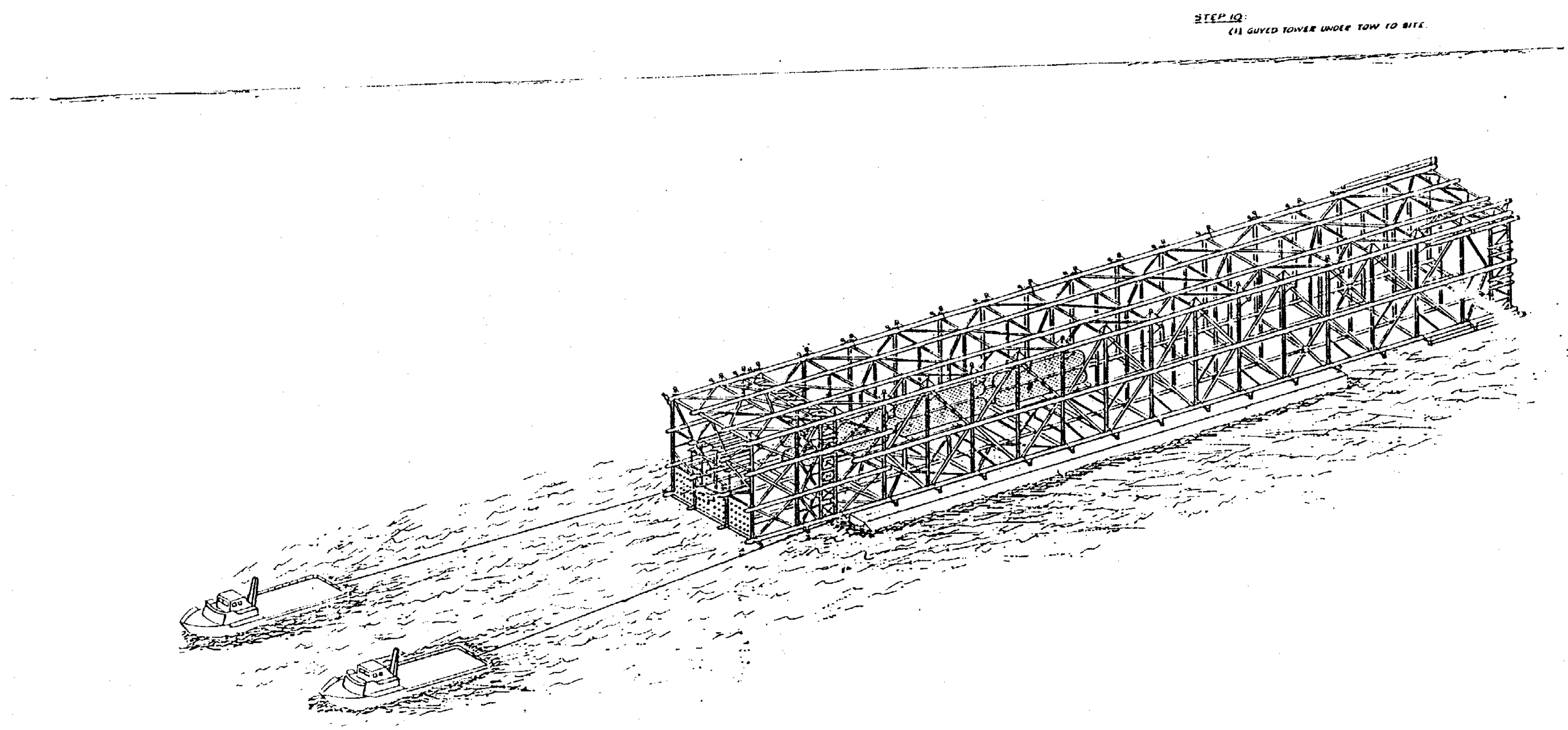
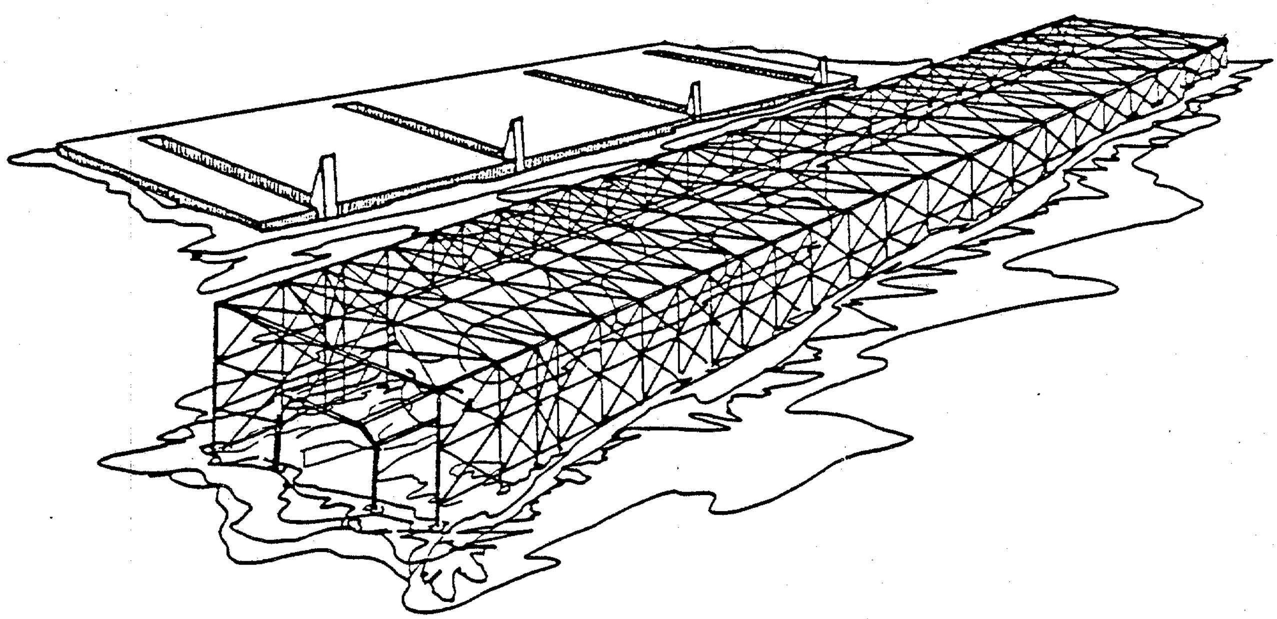
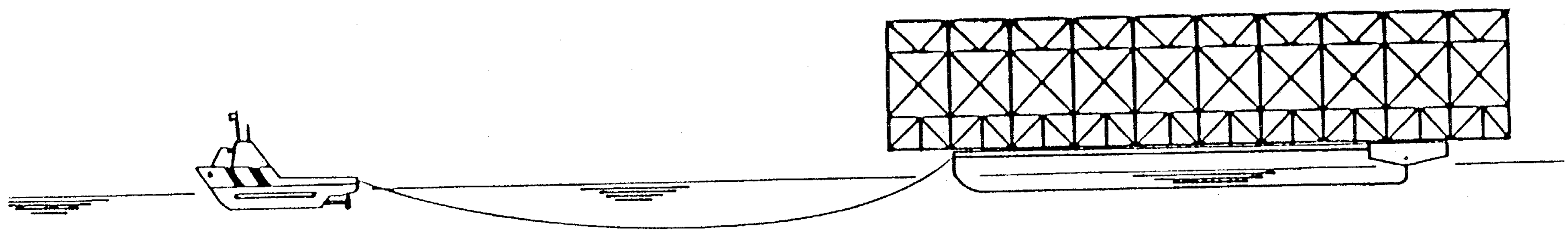


FIGURE 5-8
TRANSPORTATION



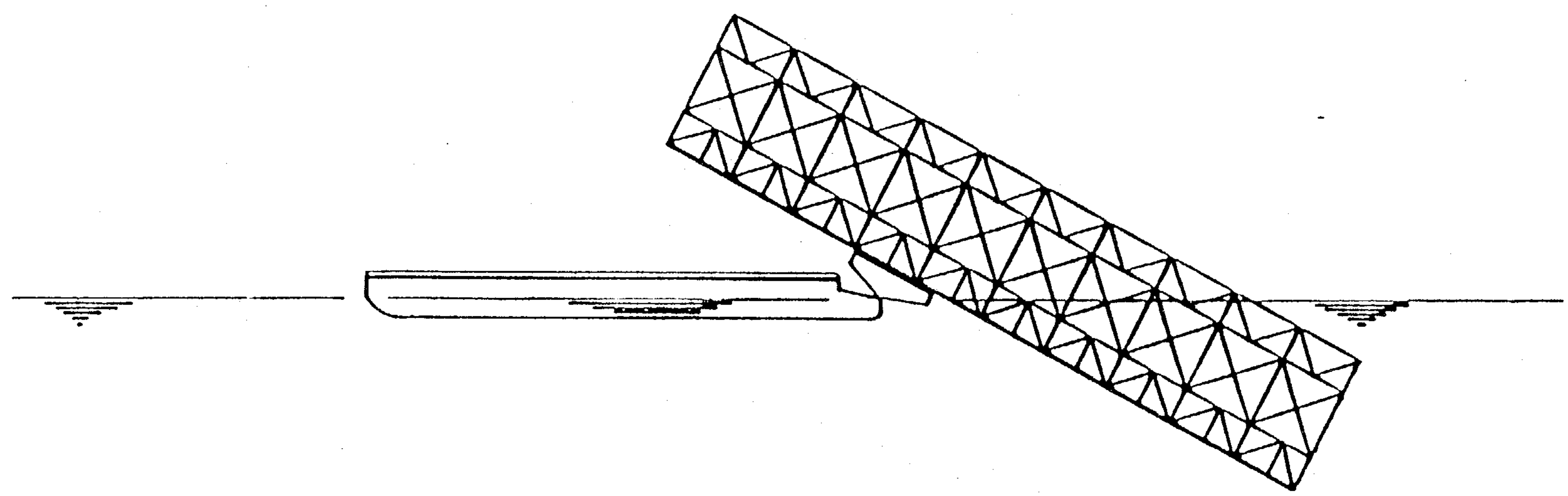
ONE PIECE INSTALLATION
SIDE LAUNCH OFF BARGE

FIGURE 5-9



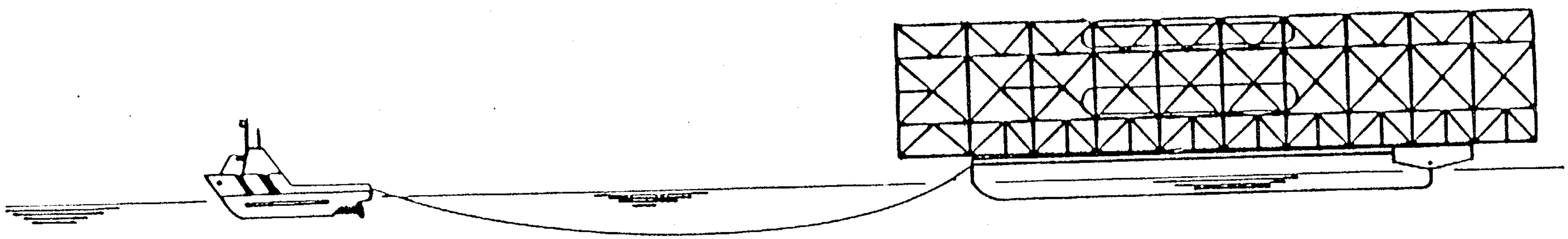
TWO PIECE INSTALLATION
LOWER TOWER TRANSPORTATION

FIGURE 5-10



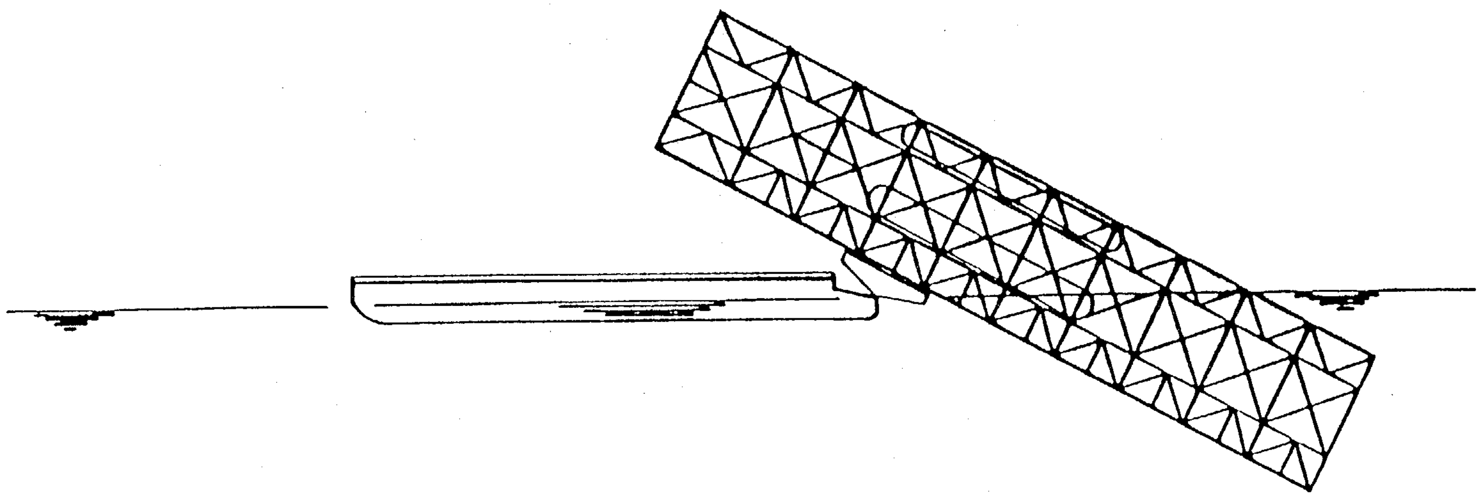
TWO PIECE INSTALLATION
LOWER TOWER LAUNCHING

FIGURE 5-11



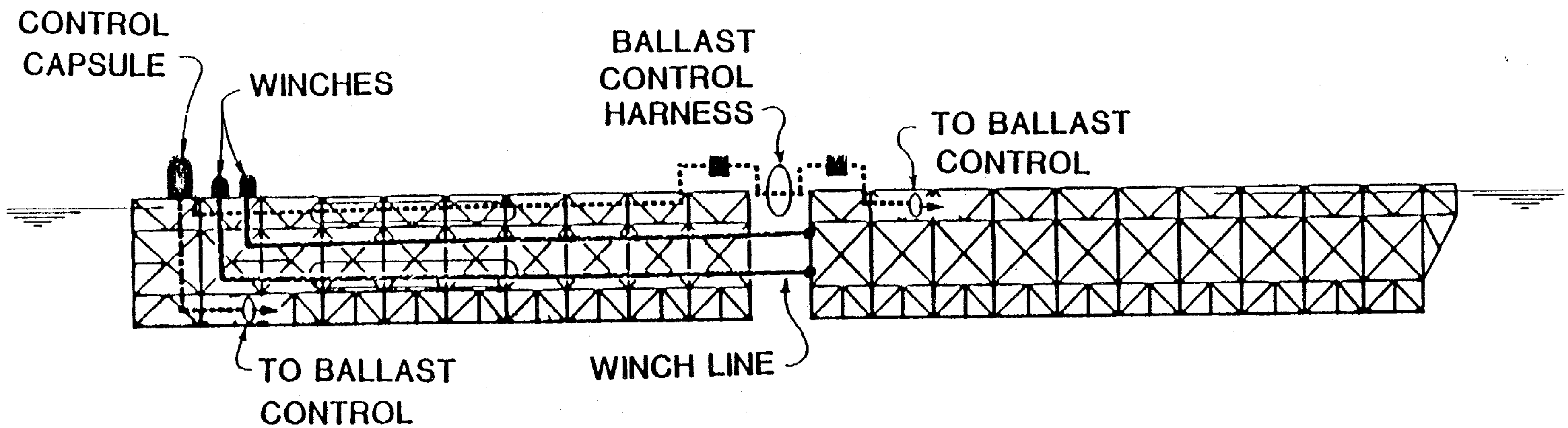
TWO PIECE INSTALLATION
UPPER TOWER TRANSPORTATION

FIGURE 5-12



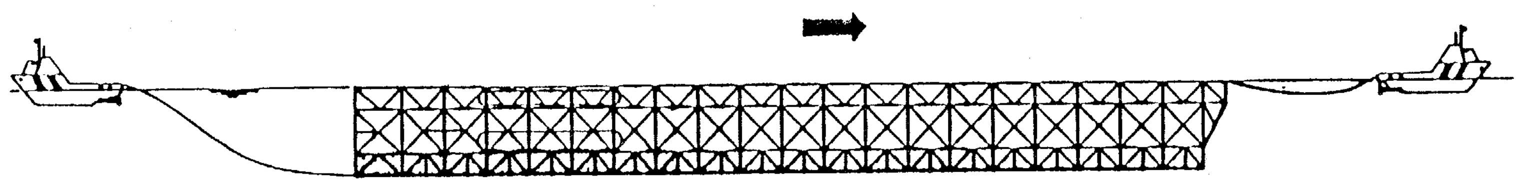
TWO PIECE INSTALLATION
UPPER TOWER LAUNCHING

FIGURE 5-13



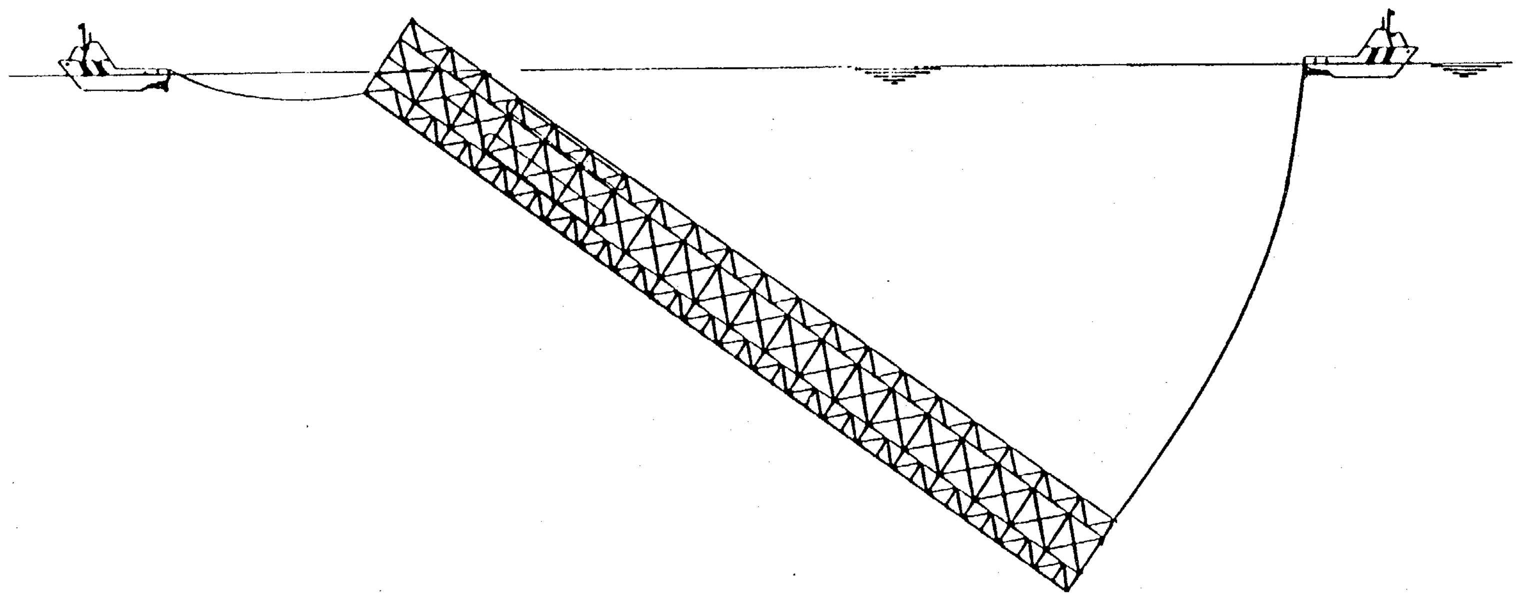
TWO PIECE INSTALLATION
TOWER MATING

FIGURE 5-14



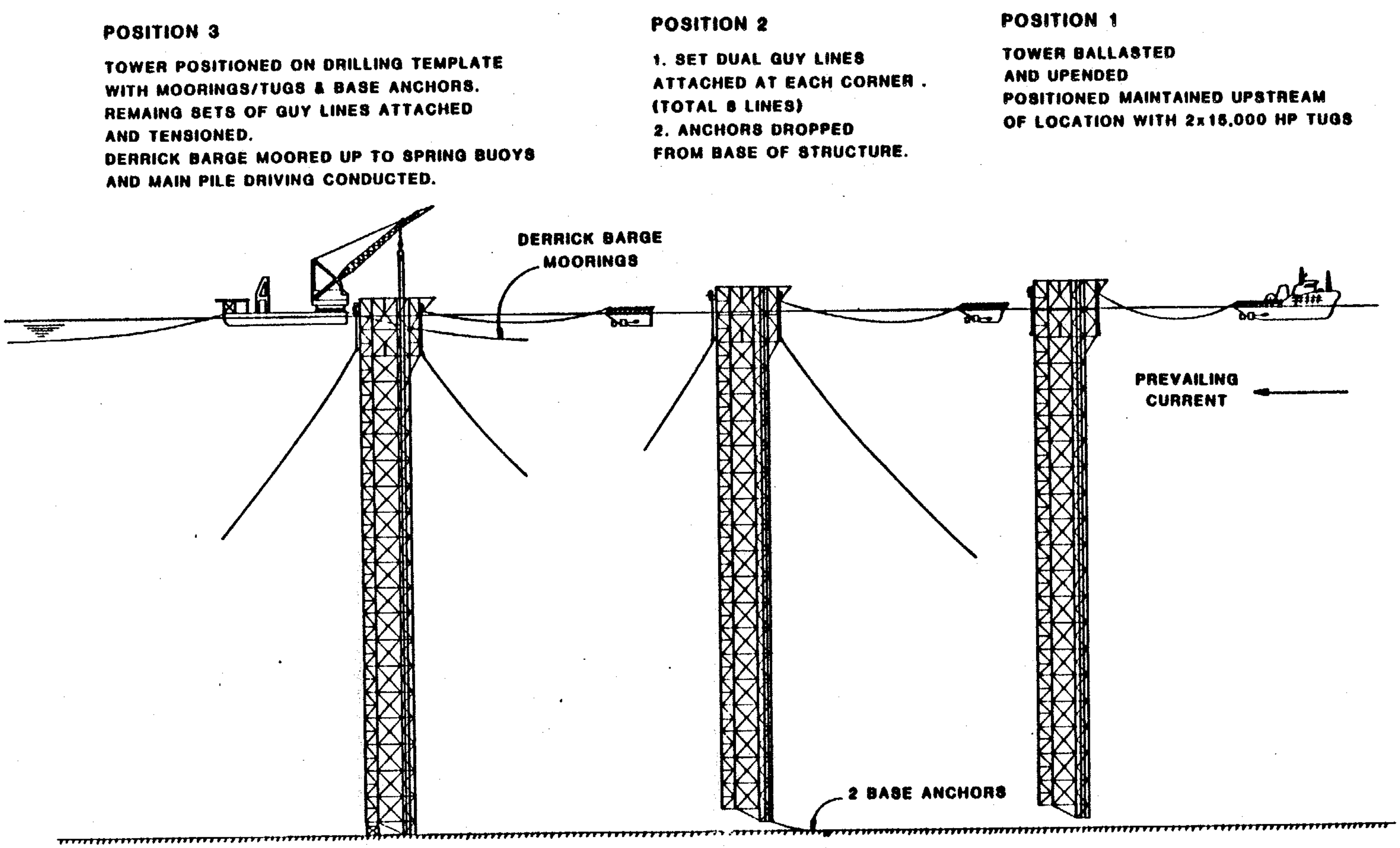
TOW TO LOCATION

FIGURE 5-15



TOW TO LOCATION

FIGURE 5-16



DETAILS OF TOWER POSITIONING AND MAIN PILE DRIVING

FIGURE 5-17

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