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LABORATORY MODEL TESTING OF BRIDGE PROTECTIVE SYSTEMS AND DEVIC--ETC(U)

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LABORATORY MODEL TESTING OF

BRIDGE PROTECTIVE SYSTEMS AND DEVICES

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16. Abstract

This report presents investigations and descriptions of a computer and laboratory model study of bridge protective systems and devices. Eight (8) bridge protective systems were run by computer analysis to ascertain their effectiveness as a protective system. In addition, two (2) bridges: the Benjamin Harrison Bridge and the Berwick Bay Bridge were analyzed by the computer and by hydraulic model testing. Based upon the analyze of these ten (10) bridges and their respective protective systems design standards and criteria were developed for AREA and AASHTO.

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OF

BRIDGE PROTECTIVE SYSTEMS AND DEVICES

(FENDERING)

DOT-CG-908665-A

September 1981

UNITED STATES COAST GUARD/WASHINGTON, D.C. 20593

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General Introduction

The changing character and volume of marine transportation presents new and increasing demands on our waterways and adjacent structures such as bridges, wharves, harbor piers, marinas, and lock entrances. These demands are attributed to the phenomenal growth in navigation module, (size and speed) whether it be a tanker, containership, or barge tows. Because of these increases in navigation module, the forces which can be delivered to structures adjacent to the waterway have substantially increased.

The increased number of collisions with bridges in recent years has focused more interest in the Coast Guard bridge replacement program. Coast Guard casuality statistics show that vessel collisions with fixed objects, such as bridges, more than doubled between 1966 and 1975 as larger and greater numbers of vessels used the nation's waterways. One Coast Guard study reveals that during the period FY 71 to FY 75, $23,153,000 in damage and fourteen fatalities were encountered. Obviously, such statistics indicate that a need exists to assure that proper design practices are used for fendering system installation. This need was recognized by the Bridge Division, U.S. Coast Guard which is charged with the responsibility to provide for the economic efficiency and safety of marine transportation under bridges spanning the nagivation roads of the United States.

Tankers built during the early part of this century had an overall length of about 90 to 150 meters (300 to 500 feet) and displacements as light as 5,000 long tons. Currently tankers are being built greater than 300 meters (1000 feet) with displacements of more than 400,000 long tons. Bulk carriers have grown from 120 meters (400 feet) to over 200 meters (650 feet) with displacements of 20,000 to 22,000 long tons. Additionally, barges moving on the Gulf Intracoastal Waterway now measure up to 90 meters (300 feet) and have a liquid capacity of 5,000,000 liters, 31,000 barrels or 3,000 long tons.

Designing a cost effective replacement bridge with a navigation prism, capable of meeting present day needs of navigation, is easy to justify. These replacement bridges are highly cost effective because the replacement bridge provides for optimum productivity of navigation. In other words, vessels of economic size are able to utilize the waterway resources. However, the protection of bridge piers is another matter. They are not unlike the car bumper which provides no utilitarian value of space or mobility to the passanger. Conceptually, protection of the vessels and bridge piers is provided by a fender system adjacent to the navigation opening of the bridge, rather than the vessel. The dilemma which now exists is, simply, that vessels have grown in magnitude and disproportionately to the growth in size or capability of the bridge protective systems. To completely protect against maximum possible impact of collision, in some cases, now dictates placements of a mass in the waterways so heavy that its cost exceeds the cost of the bridge it protects. Such a mass, however, affords little or no protection ot the vessel; as the ideal bridge protective device absorbs the energy of impact equally with the vessel. These problems remain a concern of the bridge owner, the vessel operator, port authorities and the U.S. Coast Guard. It certainly follows that full utilization of existing technology and identification of these areas where research is needed will lead to better protective systems at a lower cost.

Protective systems such as fenders obviously contribute to navigation safety by minimizing or preventing damage to vessels and structures when something goes wrong during a maneuver. While many protective systems exist with varying degrees of design criteria and standards, their application to a specific case is not widely understood by those responsible for selecting the particular system. Many factors must be considered in the design of a fendering system. Factors which should be considered (which may or may not be) include: size, contours, speed and direction of the vessel using the facility, the wind and tidal current conditions expected during the vessel's maneuvers, the rigidity and energy-absorbing characteristics of the fendering system and vessel, and finally the sub-grade soil conditions. The final design selected for the fendering system will evolve after making arbitrary limitations to the values of some of these factors.

Recognizing that a single source of references, containing all existing technical data, methodology for selection of optimum fendering devices and recommended standards was urgently needed, the Bridge Modification Branch under the direction of the United States Coast Guard advertised for contract services for a State-of-the-Art Study for Bridge Protective Systems and Devices on 25 February 1977. The research contract was awarded to the Department of Civil Engineering, University of Maryland on 15 July 1977.

The report analyzed and discussed the seven most basic types of fendering systems (floating fender of camel, standard pile, retractable, rubber, gravity, hydraulic and hydraulic-pneumatic, and the spring type) and their respective sub-classes as well as to the advantages and disadvantages of each. The report went further in that it discussed materials used, design parameters, and hand computations for the design of each system. In addition, the report discusses a computer program capable of analyzing any given fendering system for varying design parameters. The report further attributes a chapter for recommendations for future research. A statement is as follows:

> "It is obvious from the completed study that present day bridge protective systems and devices are inadequate. In other words, they are either under or overdesigned. This is attributed to the fact that tankers, containerships, and barge tows have increased in substantial size without a proportional change in design criteria or innovative ideas in bridge protection. Future research should center on the design, analysis, and laboratory modeling of old, new, and innovative ideas in

fendering. With today's inadequate bridge protective systems and devices, it would appear appropriate to select a specific bridge which has received considerable publicity because of its history of collisions, damages, and delays to navigation and conduct a model test in a laboratory. A typical example would be the Southern Pacific Transportation Company bridge across the Atchafalaya River at Berwick Bay, Morgan City, Louisiana or the Benjamin Harrison Bridge over the James River in Hopewell, Virginia."

Therefore based on the results of the State-of-the-Art Study, the U.S. Coast Guard initiated a Phase II study entitled "A Laboratory Model Testing of Bridge Protective Devices and Systems (Fenders)".

The contract (CG-908-665A) consisted of the following steps:

1. A computer laboratory model test for various bridges involved with bridge collision incidents during an accidental marine vessel impact in order to assess the performance of their existing protective systems and/or devices.
2. Development of new and/or improved bridge protective systems and/or devices based upon the computer laboratory model testing and the State-of-the-Art.
3. A laboratory model testing (hydraulic analysis) of the Southern Pacific Railroad bridge at Morgan City, Louisiana and the Benjamin Harrison Bridge at Hopewell, Virginia.
4. Based upon the first three steps and the State-of-the-Art write and/or develop new standards for possible inclusion into AREA and AASHTO specifications.
5. Issue to the U.S. Coast Guard five (5) separate reports as follows:

1. Report No. 1: Computer Laboratory Model Testing Results
2. Report No. 2: Laboratory Model Testing (Hydraulic Analysis) of the Southern Pacific Railroad Bridge at Morgan City, Louisiana.
3. Report No. 3: Laboratory Model Testing (Hydraulic Analysis) of the Benjamin Harrison Memorial Bridge at Hopewell, Virginia.
4. Report No. 4: Write Proposed AREA Design Specifications based upon reports 1, 2, and 3.
5. Report No. 5: Write Proposed AASHTO Design Specifications based upon 1, 2, and 3.

CHAPTER I: VARIOUS BRIDGES COMPUTERIZED

INTRODUCTION

Tankers, bulk carriers, cargo vessels, and barges are being built increasingly larger in recent years. They require more room to maneuver and supply greater cargo capacities than ever before. This places increasing demands on waterways and adjacent structures such as bridges, docks, harbors, piers, marinas, lock and port entrances.

Tankers built during the early part of this century had an overall length of about 90 to 150 meters and displacements as light as 5,000 long tons. Currently tankers are geing built greater than 300 meters with displacements of more than 400,000 long tons. Bulk carriers have grown from 120 meters to over 200 meters with displacements of 20,000 to 22,000 long tons. Additionally, barges moving on the gulf Intracoastal Waterway now measure up to 90 meters and have a liquid capacity of 5,000,000 liters or 3000 long tons. The velocities these vessels can attain has also increased. Because of these increases in size and speed, the forces which can be delivered to structures adjacent to the waterway have substantially increased.

Coast Guard casualty statistics show that vessel collisions with fixed objects, such as bridges, more than doubled between 1966 and 1975 as larger and greater numbers of vessels used the nation's waterways. One coast Guard study reveals that during the period FY71 to FY75, $23,153,000 in damage and fourteen fatalities were encountered. Obviously, such statistics indicate that a need exists to assure that proper design practices are used for fendering system installation. This need was recognized by the Bridge Division, U.S. Coast Guard which is charged with the responsibility to provide for the economic efficiency and safety of marine transportation under bridges spanning the navigation roads of the United States. They have since then initiated a research contract to Civil Design and Technology Corporation to conduct a study of the State-of-the-Art in Bridge Fendering Devices. As part of this State-of-the-Art, a computer program was written to analyze bridge protective devices.

FACTORS CONSIDERED IN THE DESIGN

The function of bridge fendering systems are to protect bridge elements against damage from waterborne traffic. There are many factors to be considered in the design of fendering systems including the size, contours, speed, and direction of approach of the ships using the facility; the wind and tidal current conditions expected during the ship's maneuvers and while tied up to the berth; and the rigidity and energy-absorbing characteristics of the fendering system and ship. The final design selected for the fender system will generally evolve after reviewing the relative costs of initial construction of the fendering system versus the cost of fender maintenance and of ship repair. In other words, it will become necessary to decide upon the most severe docking or approach conditions to protect against and design accordingly; hence, any situation which imposes conditions which are more critical than the established maximim would be considered in the realm of accident and probably result in damage to the dock, fendering system, or the ship.

TYPES OF BRIDGE PROTECTIVE SYSTEMS

As a result of the above factors many fendering systems have been designed and/or analyzed (1-72). These systems are of wide variety and material which vary considerably in design, fabrication, and cost. As a result of the literature survey, basically seven types of fandering systems are in existence.

These seven systems are as follows:

1. Floating Fender or Camels
2. Standard Pile-Fender System
    a) Timber-Pile
    b) Hung Timber
    c) Steel Pile
    d) Concrete Pile
3. Retractable Fender System
4. Rubber Fender System
    a) Rubber in Compression (Seike)
    b) Rubber in Shear (Raykin)
    c) Lord Flexible
    d) Rubber in Tension
    e) Pneumatic
5. Gravity Type Fender System
6. Hydraulic and Hydraulic-Pneumatic Fender System
    a) Dashpot Hydraulic
    b) Hydraulic-Pneumatic Floating Fender
7. Spring Type Fender System

In addition to the seven basic fendering of fendering systems, numerous protective cells and dolphins exist.

THEORY

General Techniques

The general response of a piling system, when subjected to a ship, is computed by removing the pile and examining its effect as a cantilever beam, as shown in Figure 1. The interaction of lateral elements, such as walers, are neglected and thus a conservative design. Two general theoretical

Equations are used by the designer, and are based on force-acceleration and kinetic energy relationships.

Force-Acceleration

The induced of applied force to the system, caused by the ships' impact is;

$$F_a = M(v_i^2 - v_f^2)/2\Delta_s \tag{1}$$

where:

M = Mass of ship

Δ_s = Deformation of system at point of impact

v_i, v_f = Initial and final velocity

The resisting force of the system is;

$$F_r = 3\Delta_s E(I/D.F.)/L^3 + \Sigma k\Delta_s \tag{2}$$

where:

E = Modulus of elasticity of pile

I = Inertia of pile

D.F. = Lateral distribution of load due to lateral stiffness of effect

L = Cantilever length of pile

k = Spring constant of fendering

The induced moment and stress is computed from;

$$M = F_a \times L \text{ and} \tag{3}$$

$$f = M/(S/D.F.) \tag{4}$$

where:

S = Section modulus.

In applying this method the designer would assume and allowable Δ_s and initial stiffness I. If the resisting force $F_r > F_a$, then the actual Δ_s would be smaller than assumed. The induced stress f would be compared to the allowable or ultimoate stress of the material.

Kinetic Energy

The induced energy caused by the ship is given by;

$$E_{in} = \frac{1}{2} Mv_i^2 (C_H)(C_s)(C_c)(C_E) \tag{5}$$

where:

v_i = Initial or translational ship velocity

C_H = Hydrodynamic coefficient = $1 + \frac{2D}{B}$

D = Draft of ship

B = Beam of ship

C_E = Eccentricty coefficient

C_s = Softness coefficient

C_c = Configuration coefficient

The C coefficients (C_E, C_s and C_c) can be set equal to 1.0 for the worst case. Other variations can be obtained for specific ship variables, as given in Ref. (1).

The output energy of the energy that can be absorbed by the piling system is;

$$E_o = F \cdot \Delta_p + \Sigma \frac{1}{2} k\Delta_f^2$$

but

$$\Delta_p = FL^3/3E(I/D.F.)$$

therefore,

$$E_o = F^2L^3/(3EI/D.F.) + \Sigma \frac{1}{2} k\Delta_f^2 \tag{6}$$

Using Equations (5) and (6) and assuming $\Delta = FL^3/3EI/D.F.$ or zero, the induced force F is determined. The resulting Δ can then be evaluated and used to re-evaluate E_o if $\Delta = 0$ was originally assumed. The resulting moment and stress if found as per Equations (3) and (4).

System Technique

A complete pile system is shown in Figure 2, and includes the support piles and lateral walers, excluding fenders. This system, in effect is a cantilever grid plate, subjected to a lateral load. The response of such a system can readily be determined by using matrix formulations or a finite difference scheme, the latter of which will be presented herein.

Consider several interacting elements of the system, as shown in Figure 3. Assuming a uniform load is applied along each member, the load deformation response given by the basic relationship;

$$\frac{d^4w}{dx^4} = \frac{q_x}{EI_x} \tag{7}$$

$$\frac{d^4w}{dy^4} = \frac{q_y}{EI_y} \tag{8}$$

where:

EI_x, EI_y = Member stiffness

W = The vertical deformation

q_x, q_y = The external applied loads in force

Equations (7) and (8) can be written in difference form (3) from the relationship;

$$\frac{d^4w}{dx^4} = (W_{\ell\ell} - 4W_\ell + 6W_o - 4W_o + W_{rr})/\lambda_x^{\,4} \tag{9}$$

$$\frac{d^4w}{dy^4} = (W_{aa} - 4W_a + 6W_o - 4W_b + W_{bb})/\lambda_y^{\,4} \tag{10}$$

where the nodes relative to each deflection point is prescribed along the two girders and spaced uniformly, as shown in Figure 4.

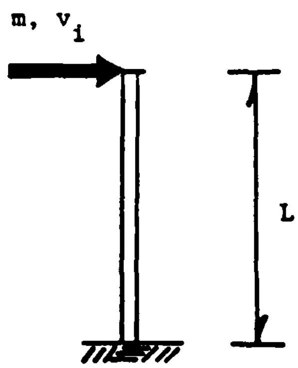


FIG 1-1 **Isolated Pile**: Cantilever
(Undergoing Vessel Impact at Vessel mass m,
and Vessel Velocity V_i)

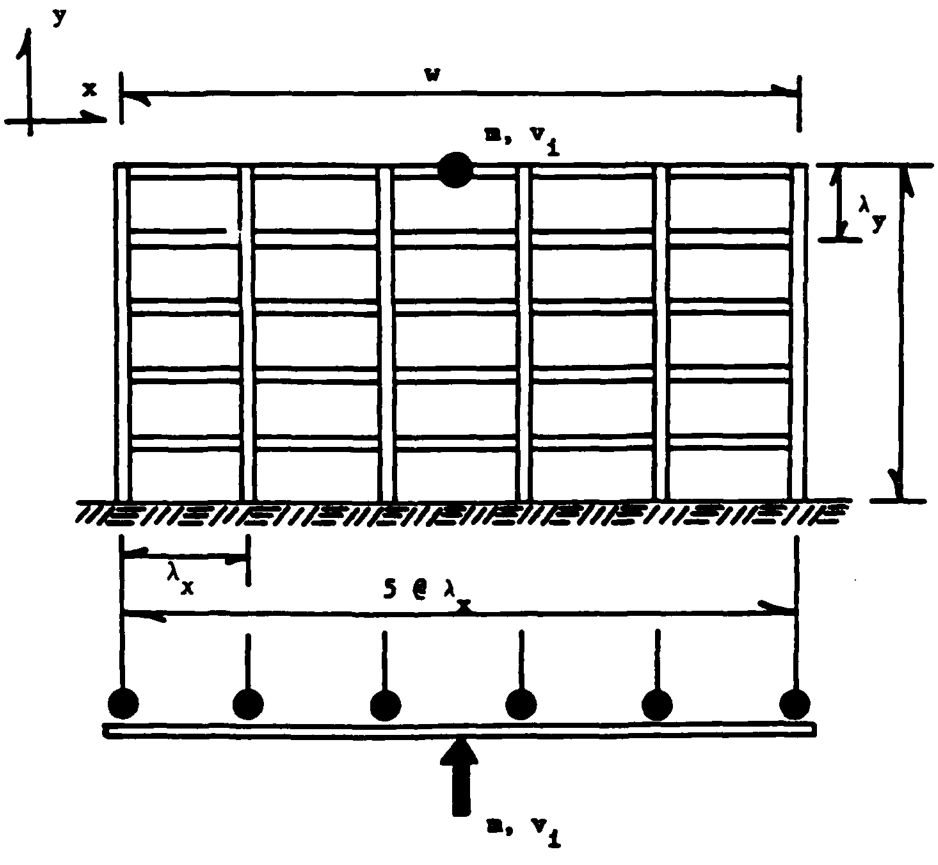


FIG 1-2 **Fendering System** (Vertical Piles with Horizontal walers Attached)

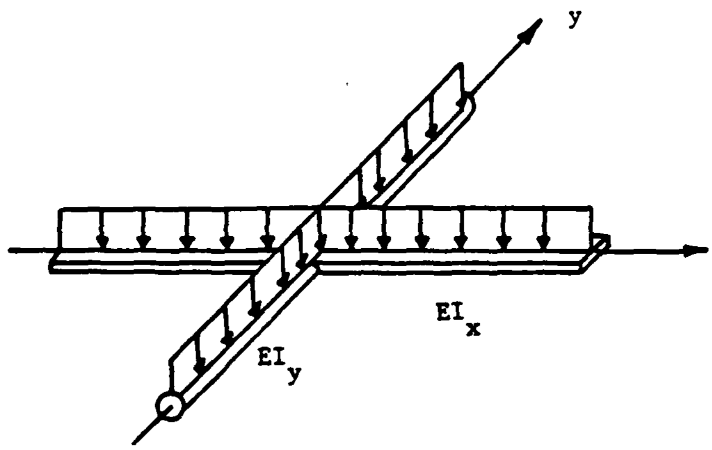


FIG 1-3 **Intersecting Elements** (Pile with Waler attached showing Stress Distribution)

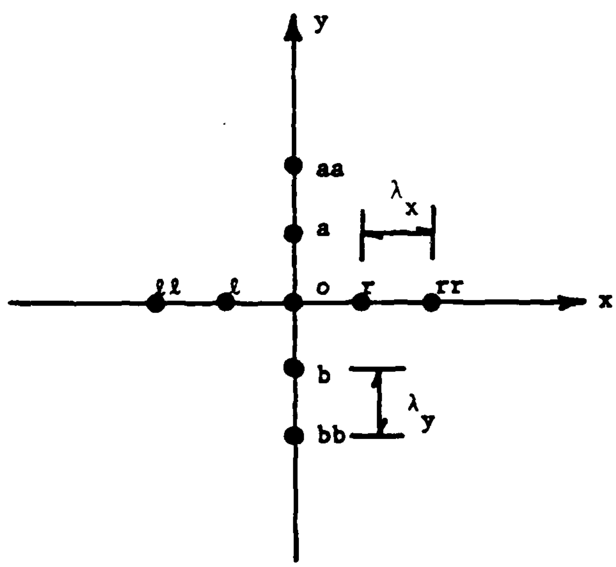


FIG 1-4 **Nodal Identification** (of Intersecting Elements)

Assuming now the total applied load on the grid is q (force per unit area), then the resistance is proportional to;

$$\frac{q_x}{\lambda_y} + \frac{q_y}{\lambda_x} = q \tag{11}$$

Substituting in Equations (9) and (10) into (7) and (8) gives q_x and q_y, and then substituting into Equation (11) gives;

$$\begin{aligned} &D_x/\lambda_x^{\,4}(W_{\ell\ell} - 4W_\ell + 6W_o - 4W_r + W_{rr}) \\ &+ D_y/\lambda_y^{\,4}(W_{bb} - 4W_b + 6W_o - 4W_a + W_{aa}) = q \end{aligned} \tag{12}$$

where:

$$D_y = EI_y/\lambda_x,\; D_x = EI_x/\lambda_y \tag{13}$$

Defining $D_x = \alpha D_y$ and $\lambda_x = n\lambda_y$ and substituting into Equation (10) gives the resulting mesh Equation (14)

$$\left|\begin{array}{ccccc} & & n^4 & & \\ & & -4n^4 & & \\ \alpha & -4\alpha & 6(\alpha + n^4) & -4\alpha & \alpha \\ & & -4n^4 & & \\ & & n^4 & & \end{array}\right| \cdot W = \frac{qn^4\lambda^4 y}{D_y} \tag{14}$$

Equation (14) represents the general load-deformation response of the grid when subjected to a uniform load q. In order to apply this equation to the cantilever plate, appropriate boundary conditions must be applied. For the basic cantilever plate, the free edges have boundary conditions M = V =) and along the fixed edge W = θ = 0, where;

M = Bending moment

V = Shear face

W = Deflection

θ = Slope

These modifications, considering all possible conditions along the plate, result in a total of twelve cases, including the general case given by Equation (14) whose locations are shown in Figure 5.

All of these cases and their resulting equations have been programmed for direct evaluation of the deformation of the plate for any stiffness and loading. The application of this program will now be described.

PARAMETRIC STUDY

As illustrated by the general theoretical techniques, the distribution factor is important if it is desirable to determine the system response. The determination of this factor (D.F.) has been obtained for typical grid stiffnesses (D_x, D_y) and span length (L) or height of the pile. A unit load effect was used in examining the system and single pile.

Longitudinal Stiffness (D_y)

The range in the stiffness $D_y = \frac{EI_y}{\lambda_x}$ was determined by examination of typical stell HP, steel W, and 12 to 28 in. (30.5 to 71.1 cm.) round timber members which are used in piling systems. The spacing λ_x was varied between 5 ft. (1.5 m.) and 25 ft. (7.6 m.), in five feet (1.5 m.) increments. The length of cantilever plate was varied between 20 ft. (6.1 m.) and 60 ft. (18.3 m.) in four feet (1.2 m.) increments.

Transverse Stiffness (D_x)

The range in the stiffness $D_x = \frac{EI_x}{\lambda_y}$ was determined by also examining typical walers, which consisted of steel W and 10 in. (25.4 cm.), 12 in. (30.4 cm.), and 14 in. (35.6 cm.) timber sections. The spacing λ_y was varied the same as the λ_x variable.

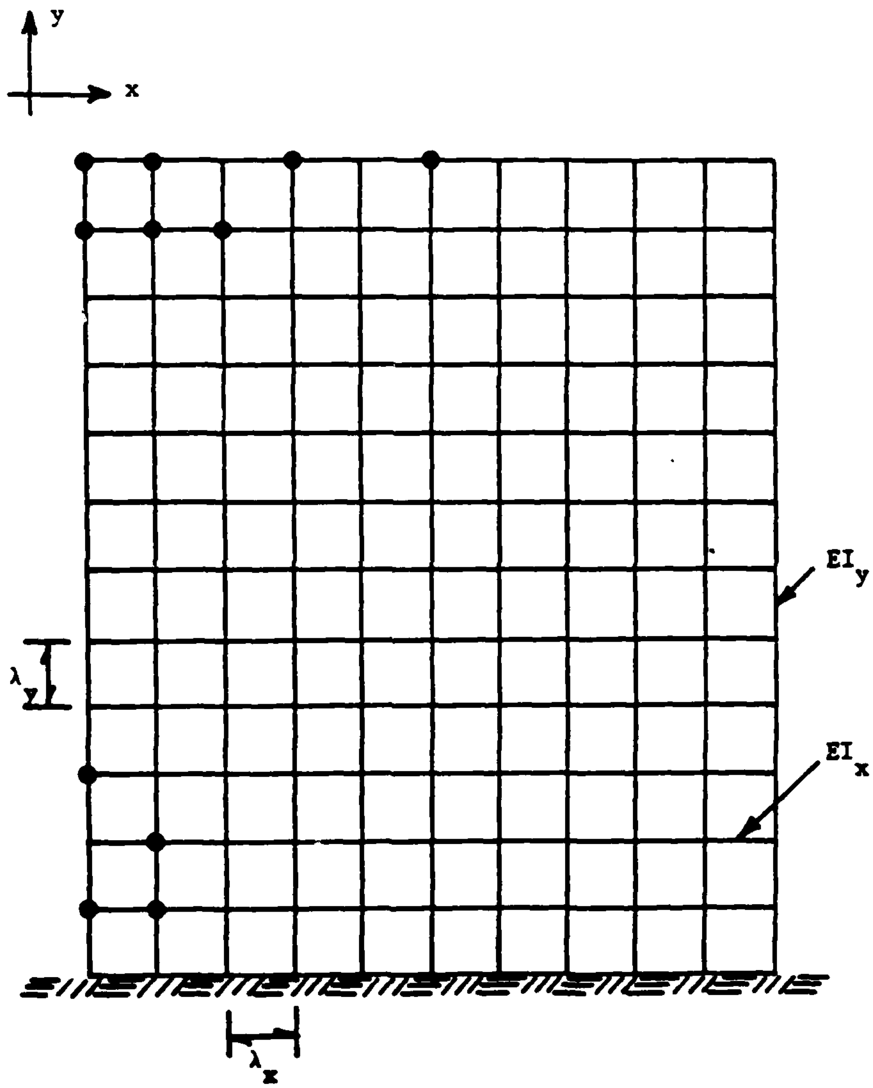


FIG 1-5 Grid Case (Piles with Walers)

Range in Parameters

A study of all of the resulting stiffnesses indicates the following ranges;

Variable	Lower Bound	Upper Bound
D_y	2×10^4 K-in. (231,000 Kg-m.)	6×10^5 K-in. (7,000,000 Kg-m.)
D_x	4×10^3 K-in. (41,000 Kg-m.)	10×10^4 K-in. (1,150,000 Kg-m.)
L	20 ft. (6.1 m.)	60 ft. (18.3 m.)

Grid Difference Solutions

Using these ranges in parameters and applying a unit load, the maximum deformation in the system has been obtained. In all these solutions a maximum of ten vertical mesh lines was used, where the spacing of the lines was set equal to a constant λ_x = 60 in. (152.4 cm.), which gave a width of 45 ft. (13.7 m.). The mesh points along these vertical lines was fixed at λ_y = 48 in. (122 cm.) for the range of 20 ft. (6.10 m.) to 60 ft. (18.3 m.).

The solution of systems has given the Δ_{sys} which was then divided by the factor $L^3/3EI_y$, which is called distribution factor (D.F.). These results were then plotted, (D.F.) vs. pile height L, as given in Figures 6 through 13. This ratio, given D.F., will now be described.

Distribution Factor

The finite difference cantilever grid equations can provide direct deformation values along any pile. Depending on the lateral stiffness (D_x), the deformation at the top or free edge of the piles can vary dramatically. This variation is quite important if the designer wishes to properly identify the interaction between the piles and isolated pile. A convenient method to describe such interaction is to relate the deformation of the systems (Δ_s) to that of the isolated pile (Δ_p), which is called a distribution factor of;

$$D.F. = \frac{\Delta_s}{\Delta_p} \tag{15}$$

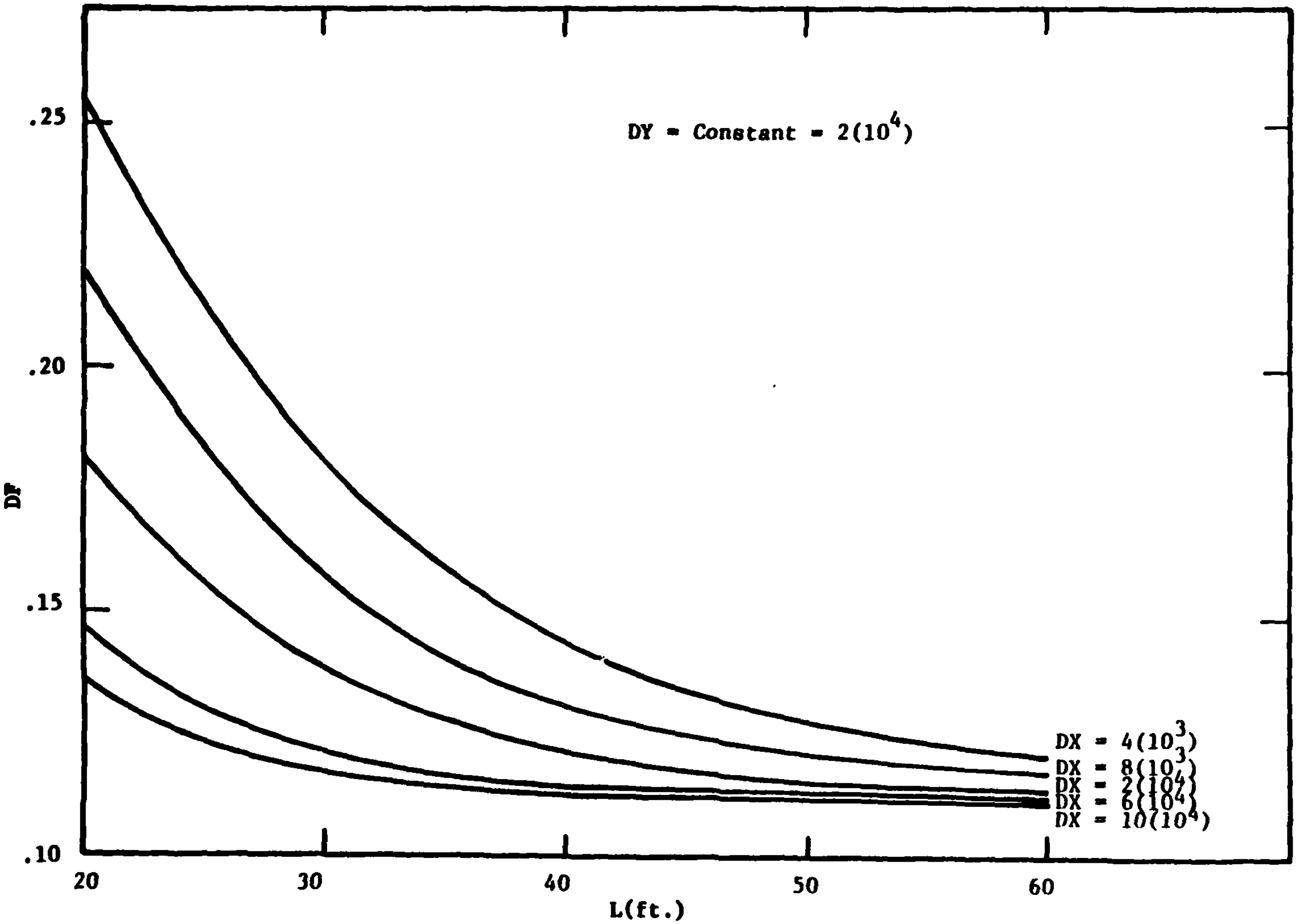


FIG 1-6 Distribtuion Factor $(D_y) = 2 \times 10^4$ k-in.

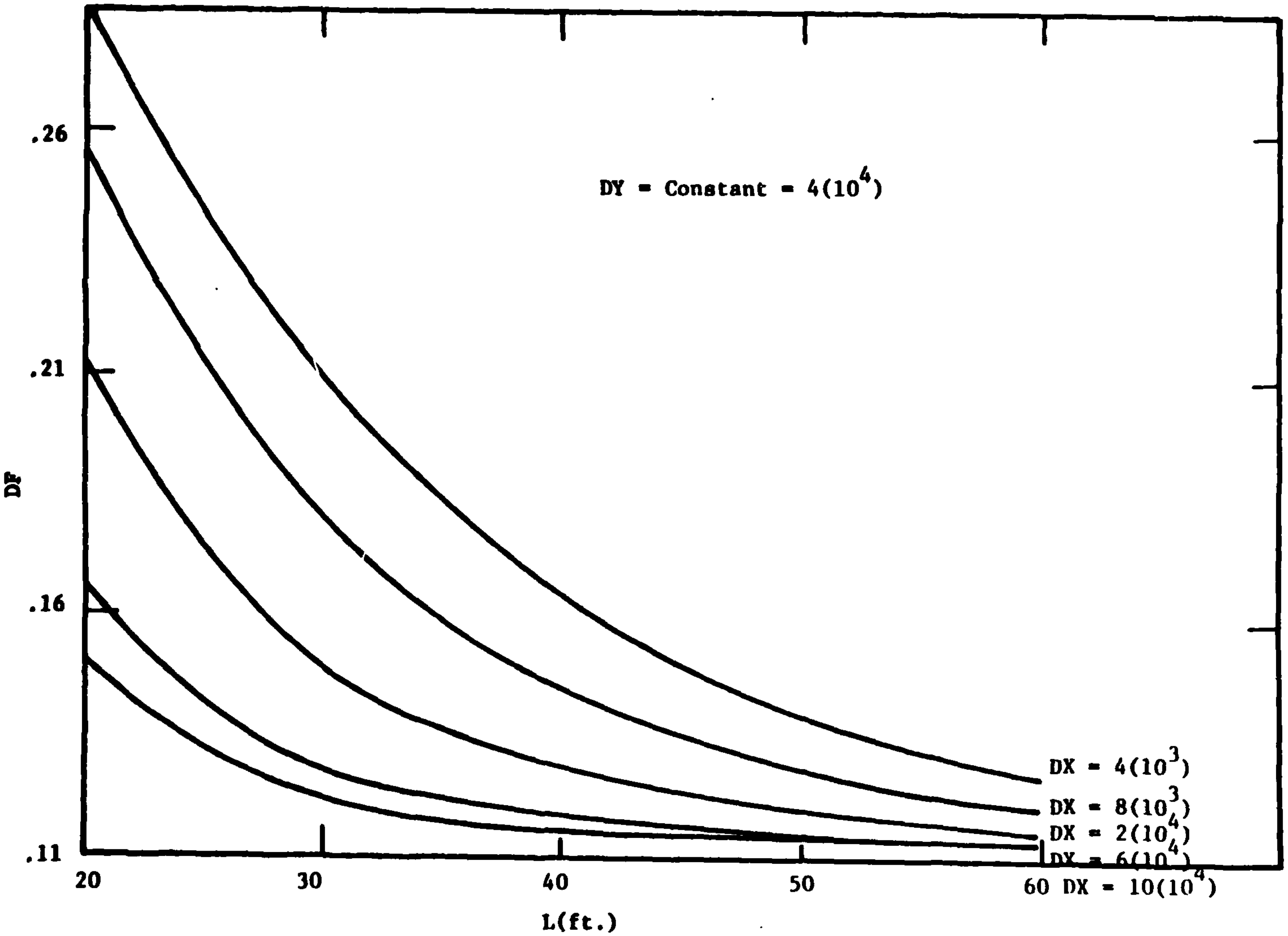


FIG 1-7 Distribution Factor (D_y) = 4×10^4 k-in.

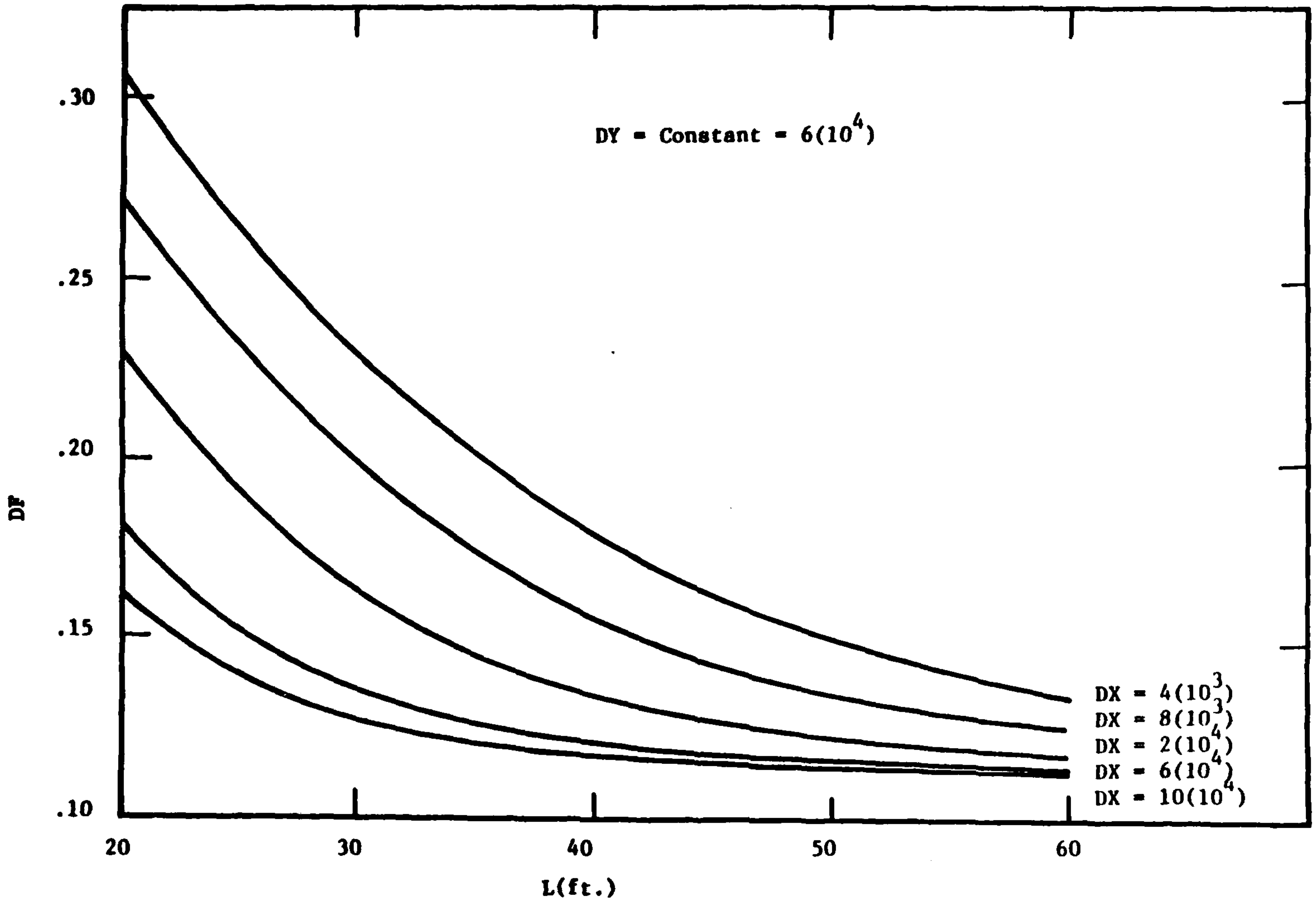


FIG 1-8 Distribution Factor $(D_y) = 6 \times 10^4$ k-in.

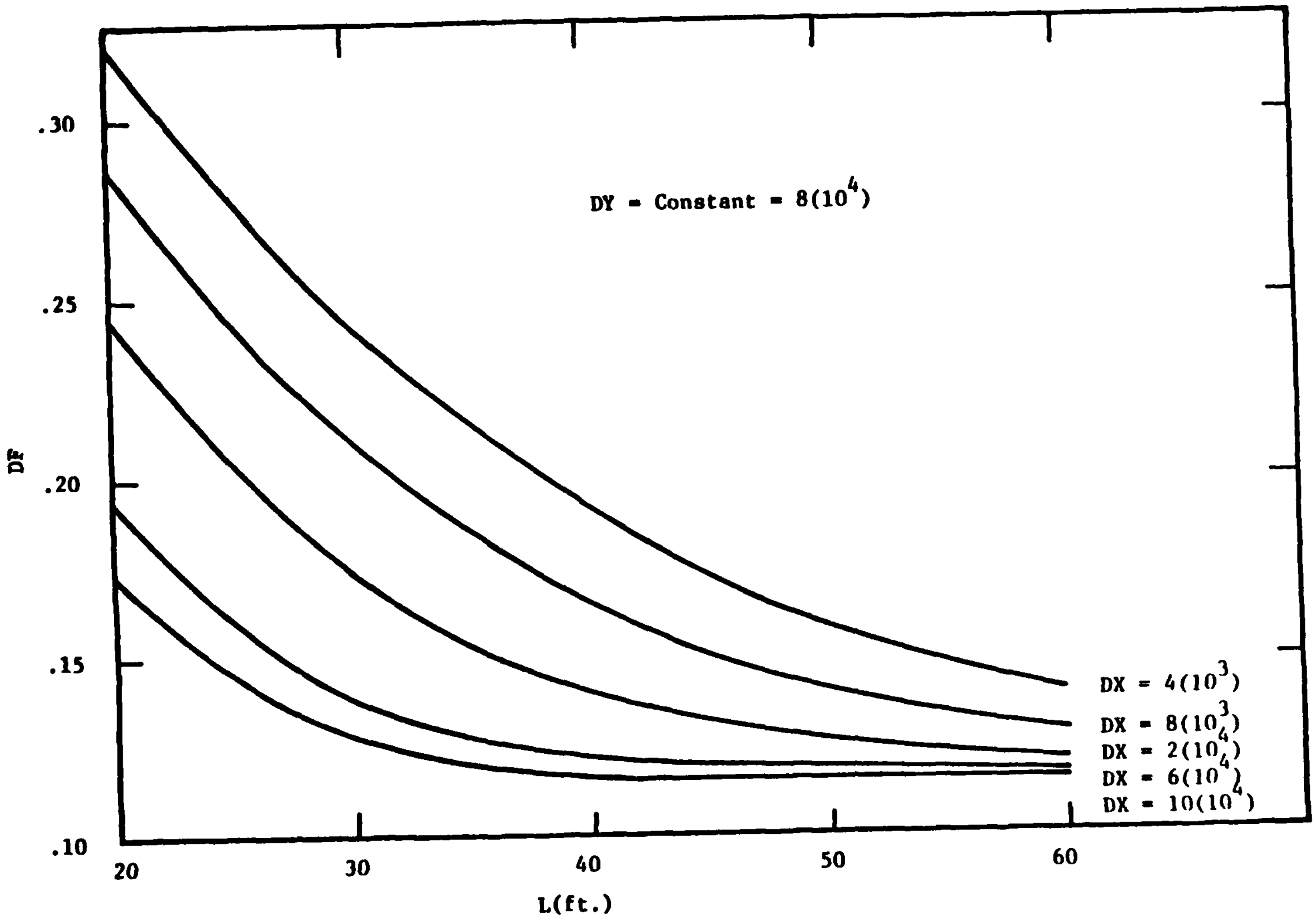


FIG 1-9 Distribution Factor (D_y) = 8×10^4 k-in.

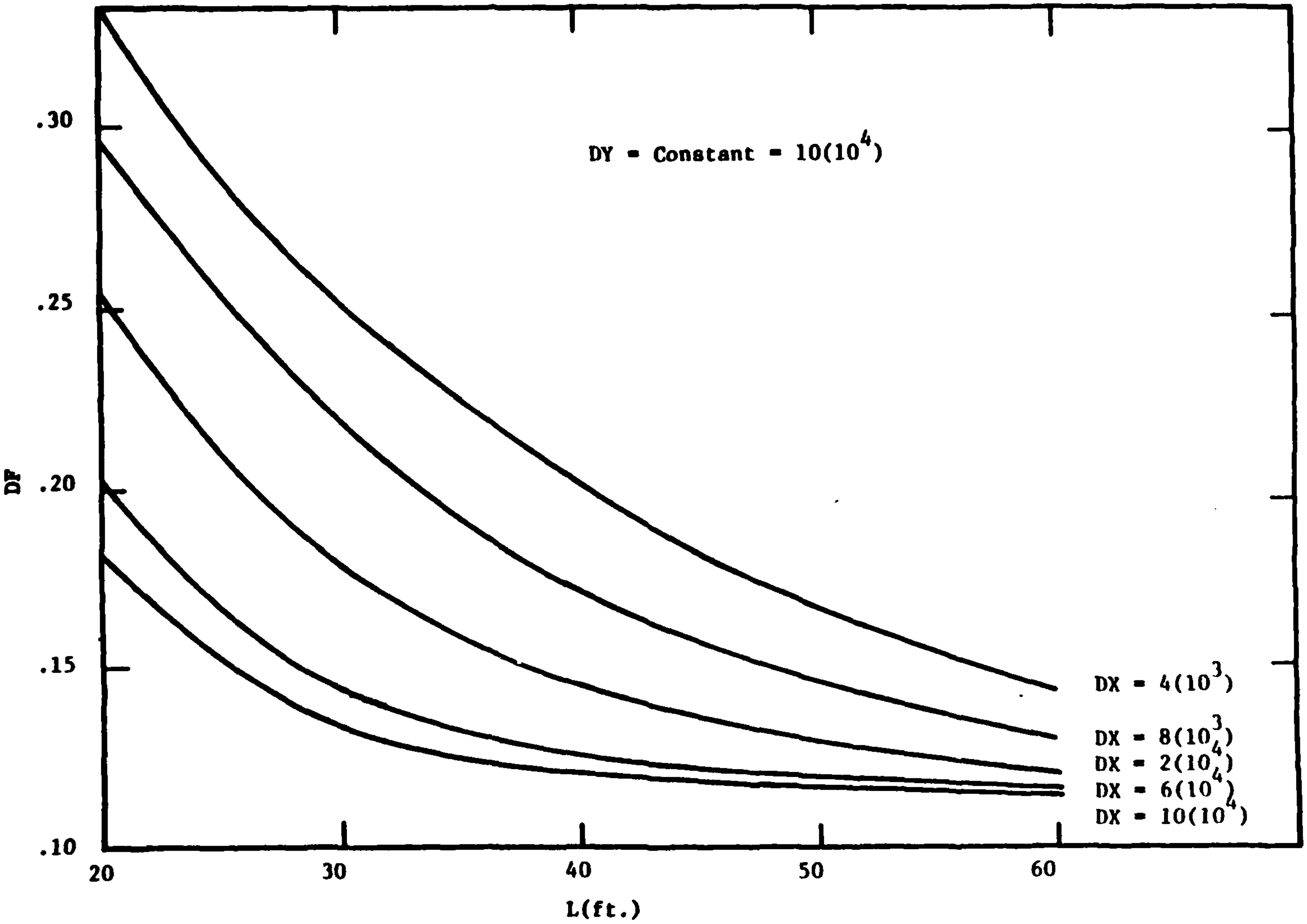


FIG 1-10 Distribution Factor (D_y) = 10 x 10^4 k-in.

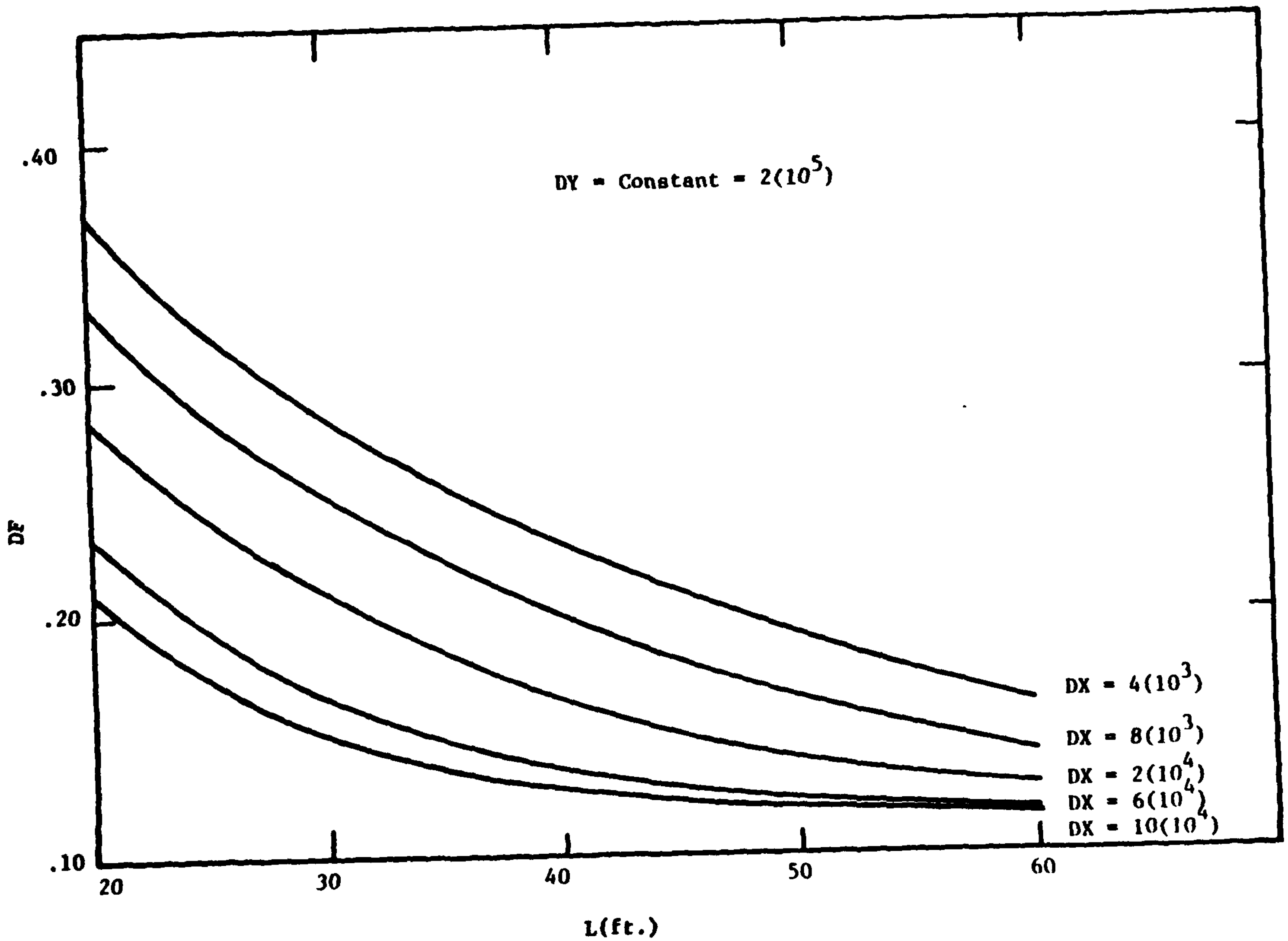


FIG 1-11 Distribution Factor (D_y) = 2×10^5 k-in.

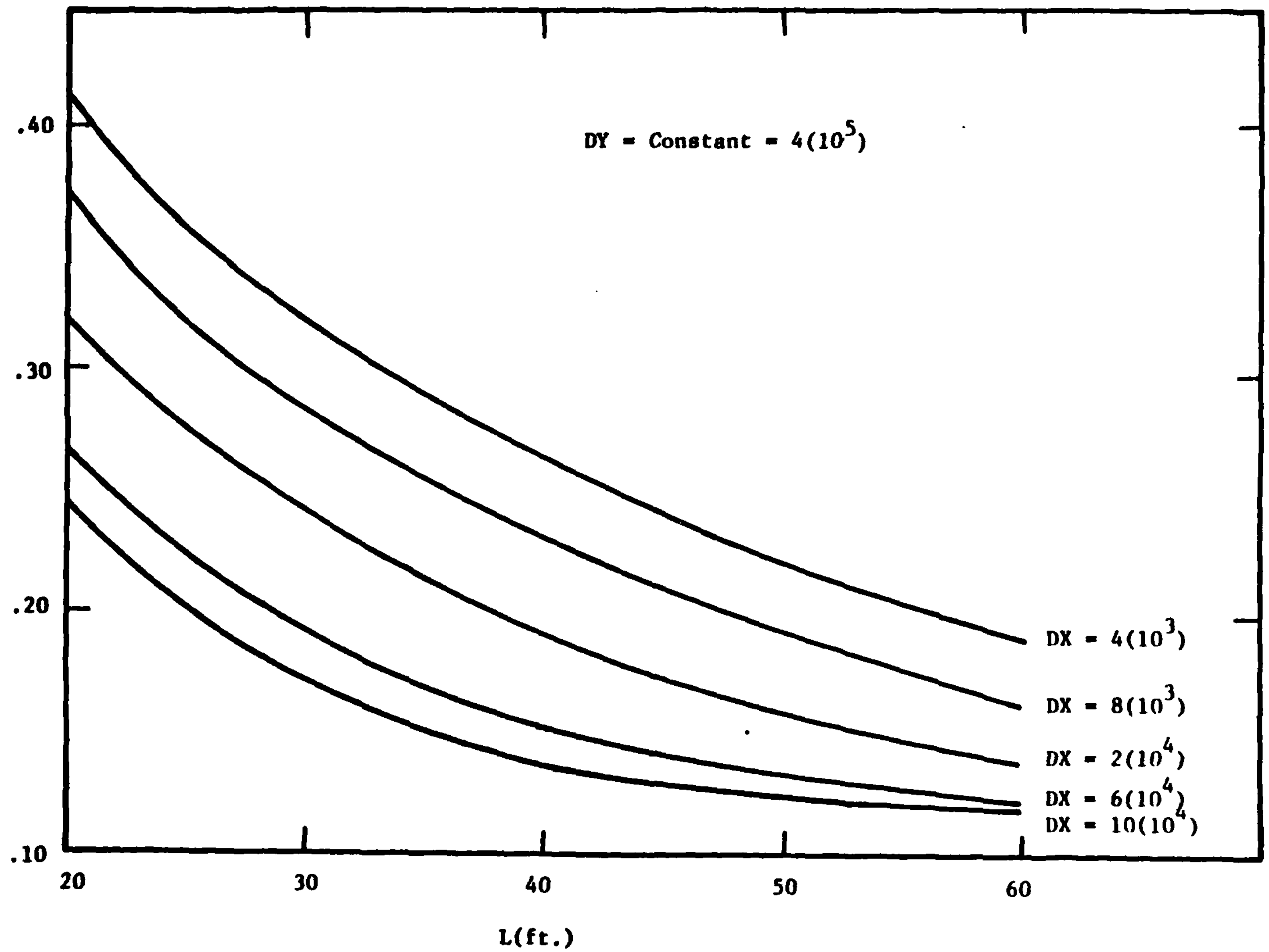


FIG 1-12 Distribution Factor (D_y) = 4 x 10^5k-in.

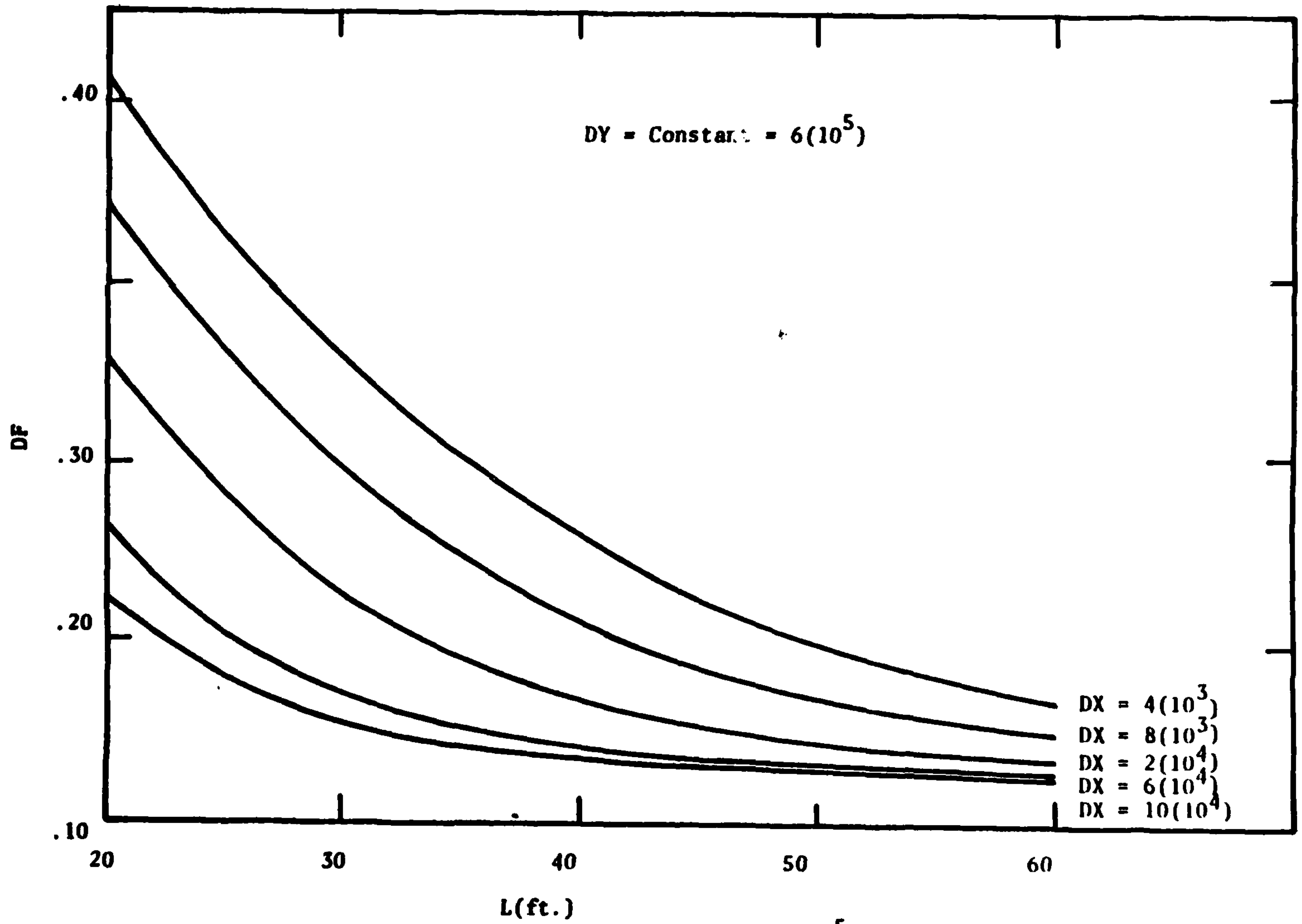


Fig 1-13 Distribution Factor (D_y) = 6 x 10^5k-in.

where:

Δ_s = Maximum deformation in the grid system (finite differences)

$\Delta_p = PL^3/3EI_y$ = Cantilever pile deformation

Equation (15) signifies the reduction in deformation of an isoloated pile when that pile is part of the system and thus the influence of lateral stiffness. Therefore, the stiffness (I) of an isolated pile can be increased by the amount of 1/D.F. or $I_{sys} = I_p/D.F.$ This factor has been referenced in Equations (2) and (6).

The resulting distribution factors, for various relative stiffnesses, are given in figures 6 through 13, and can be used for direct design.

COMPUTER ANALYSIS ASSUMPTION

Ten assumptions are necessary to utilize the existing program and they are as follows:

1. The piling interaction with the soil medium is considered; i.e., flexible supports.
2. The soil may be layered.
3. Piling group is considered as a three dimensional unit.
4. Interactions of the horizontal walers are considered.
5. Forces and deformations throughout all piles, at any time interval can be evaluated.
6. The forces and deformations are evaluated along the length of each pile.
7. Rigid wharfs, fenders, dolphins, or combinations can be considered.
8. During ship impact, any pile that fails is noted and the system is re-evaluated.
9. Total energy in the system, input and output, is computed during each time interval.
10. The system may have any general plan orientation; i.e., straight curved, etc.

COMPUTER PROGRAM

The computer program utilized was written on the UNIVAC 1108 computer and written in the FORTRAN IV language. As indicated above, this program has the capability of analyzing any given bridge protective system and/or device.

The basic theory utilized in a protective device system consists of several sub-systems. One sub-system consists of complete interaction of the supporting piling systems, which includes any member of piles, pile types, and soil characteristics. The other sub-system is the interaction of the system, supports, fenders (if applicable) and any distribution beams. This entire system is then examined under the impact of the vessel, at any attached angle. At any instance, the piling is examined for a failure mode. When a given pile fails, the system is automatically modified and the dynamic analysis is continued. Automatically this process is continued until the vessel stops or all the energy is consumed, i.e., failure of all piles. At each instant of a pile failure, the resulting forces and stress on this failed pile is listed.

Input consists of the size (tonnage), contours, speed and direction of approach of the vessel, rigidity and energy absorbing characteristics of the protective system and of the vessel, the soil parameters, and finally, the geometry and size of the protective system and the materials used.

Output includes the velocity of the vessel at any instance and the load deformation of the protection. Further, it gives the energy absorbed by the protective system and the vessel's at any distance.

The results are then interpreted as to whether the protective system is adequate for the given conditions or whether it is over or under designed. If the proposed protective system is found to be under designed for the given conditions, then a major catastropic failure may be avoided. Recommendations can then be made as to what structural elements to increase in size.

If the results are interpreted to be over designed, then recommendations can be made to decrease the size of the structural elements.

In either the under or over designed case, dollars are saved, lawsuits, are avoided (underdesigned) and materials may be saved (overdesigned).

BRIDGES ANALYZED

Based upon the results of the State-of-the-Art and the potential for the use of the computer program the U.S. Coast Guard under the direction of Mr. R.T. Mancill, Jr. (Chief, Bridge Modification Branch) initiated a Phase II study entitled "A Laboratory Model Testing of Bridge Protective Devices and Systems (Fenders)".

The contract (CG-908665-A) called for in part a computer laboratory model test for various bridges involved with bridge collision incidents during an accidental marine vessel impact in order to assess the performance of their existing protective systems and/or devices. The bridges selected to be studied by the computer method were as follows:

1. Napa Valley River Bridge (Southern Pacific Transportation Co. Bridge)
2. Third Street Bridge
3. Steamboat River Bridge
4. Holguim River Bridge
5. Schuykill Bridge
6. Connecticut River Bridge
7. Mare Island Navy Yard Bridge
8. Duwamish River Bridge
9. Berwich Bay Bridge (Southern Pacific Railroad Bridge at Morgan City, La.)

10. Berwick Bay Bridge (Southern Pacific Railroad Bridge at Morgan City, La.)

11. Berwick Bay Bridge (Southern Pacific Railroad Bridge at Morgan City, La.)

12. Benjamin Harrison Memorial Bridge at Hopewell, Virginia.

These Fendering systems of the Various bridges were tested under the following criteria:

1. Various sizes of vessels now using the waterways;

2. Various velocities and direction of approach (of the worst case);

3. Various winds (as incorporated into the velocity);

4. Various tidal currents (as incorporated into the velocity);

5. Existing soil characteristics;

6. The rigidity and energy absorbing characteristics of the fendering system and the vessel.

The performance characteristics of these fendering systems were measured in terms of:

1. Velocity of vessel at every instance of impact;

2. Load deformation of the protective system at every instance or impact;

3. Energy absorbed by the protective system and the vessels at every instance of impact.

As far as discussions are concerned, in the report, it will be limited to the first eight (8) bridges and their respective protective systems and/or devices. The results of the Southern Pacific Railroad Bridge at Morgan City, Louisiana and the Benjamin Harrison Memorial Bridge at Hopewell, Virginia will be discussed in separate reports.

MODELING ASSUMPTIONS

General:

In each of the bridges analyzed by use of the computer program they were done so with and without fenders. The velocities chosen (at the time of impact) were 1,3 and 5 knots. These velocities were assumed to take into account wind and tidal currents. The weight of the vessels were chosen as 1000, 10000, and 10000 tons. These weights were assumed to be in the realm of the majority of vessels using the waterway. In addition, all types of fendering systems were investigated.

Specific:

In addition to the above general data that had to be inputed into the computer program; specific data, dealong with material properties, pile data, and system data had to be also inputed where appropriate.

Material properties included the subgrade modulus, modulus of regidity (of the material) in which the vessel may strike, and finally the modulus of elasticity (of the material).

Pile data (if appropriate) include total length, cantilever length, slope if battered, spacing between piles, pile projection in the X and Y direction, moments of inertia in the X and Y direction, polar moment of inertia, cross-sectional area, Yield stress, neutral axis to outer fiber, vertical load, and the length of the battered pile.

System data (if applicable) include the number of pile groups, number of fenders, the a, b, and c parameters of the fender curve (k-Δ), spacing between pile groups, support beam modulus of elasticity, support beam moment of inertia, support beam area, fender beam modulus of elasticity, fender beam moment of inertia, and the fender beam area.

In the rare case in which one deals with ridid cap situations then addition general pile data need be inputed. This includes elevated platform numbers, number of fixed pile caps, number of end bearings, number of vertical piles, and number of sample points.

Generated Data:

In viewing the general and specific details of the previous sections it becomes obvious that constuction plans are necessary to generate the data.

A set of construction plans for the various bridges and their protective systems were obtained from the United States Coast Guard for each bridge under consideration. From the plan view and the structural details the bridge pier (s) and/or the protective system (s) were modeled. Once the system was modeled the general and specific information was generated and inputed into the computer program for processing.

The plan view, structural details, and modeling details for each bridge analyzed appear in Appendix A as follows:

Bridge	Figure Numbers
Southern Pacific Transportation Co. Bridge	1-14 to 1-19
Third Street River Bridge	1-20 to 1-22
Steamboat River Bridge	1-23 to 1-25
Holquim River Bridge	1-26 to 1-28
Schuylkill River Bridge	1-29 to 1-30
Connecticut River Bridge	1-31 to 1-35
Mare Island Navy Yard Bridge	1-36 to 1-39
Duwamish River Bridge	1-40 to 1-43

Appendix B contains the tabulated data (Tables 1-1 to 1-10) obtained from the plan views, structural details, and modeling details for each of the above mentioned bridges.

Load Versus Deflection Curves:

Appendix C contains the load versus deflection curves (Figures 1-44 to 1-62) for the various bridges analyzed. On each load versus deflection curve two (2) curves are plotted. One curve is the load versus deflection for the system without protective systems and the other load versus deflection curve is for the system with protective systems (fenders). Each curve is than given a k (Load/Deflection) value. Thus, utilizing the load versus deflection curve of the Napa River Bridge (Figure 1-43) for a vessel velocity of 1 knot and a mass of 1000 tons, k of the without protection (fenders) would be 271.69 kips/in. With protection (fenders) the k reduces to 11.80 kips/in. Therefore, the load delievered to a structure without protection would be 20 times greater than to one with protection for the same deflection.

These k values were computed intermally by the computer program and are thus, utilized to determine if the results of the protective system are adequate. In otherwords utilizing the k values through a mathematical relationship the protective system can be rated as to whether or not it is adequate.

RESULTS

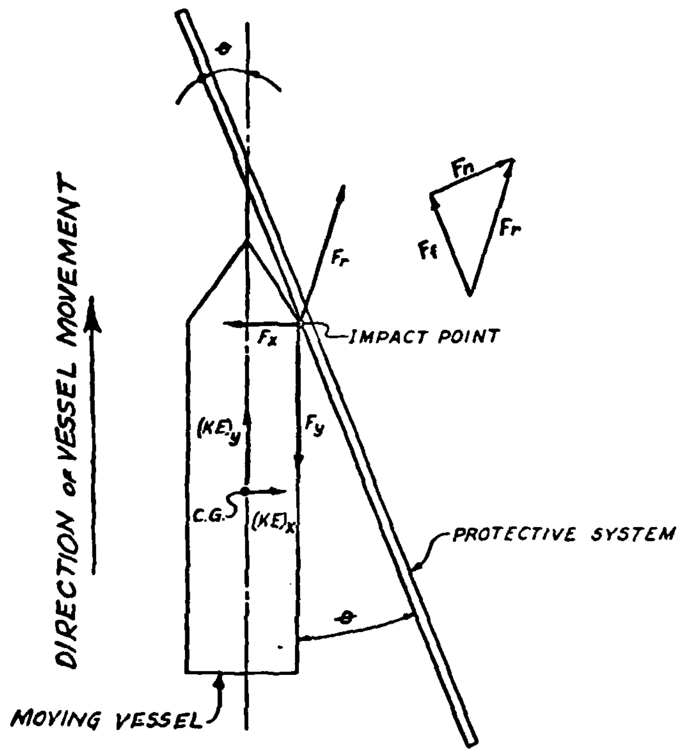
The results of the computer analysis are shown for each bridge in Tables 1-11 through 1-20. Table 1-11 shows the Induced Maximum Pile Stresses (kips per square inch). Viewing the induced maximum pile stresses and comparing it to the ultimate stress of the system will determine whether the system is adequate or not.

Tables 1-12 through 1-20 give a summary of the energy data as well as a comment as to whether or not the system failed with or without fenders.

DISCUSSION OF THE RESULTS

In utilizing the summary of energy data tables; only, one bridge (Napa River Bridge) will be discuss. After a discussion is presented it should be obvious to the reader the theory behind the results of the remaining bridges analyzed.

Table 1-12 shows the effects of a head on vessel collision with a fixed (Fig. 1.63) structure (with and without fenders). The vessel is traveling at a velocity of 1 knot and weighs 1000 tons at the time of impact. In viewing the Napa River Bridge the energy delivered to the unprotected structure would be 1095.75 kip-inches (in which k = load/deflection = 271.69 kips/inch). However, if protection were provided (in the form of fenders) the load delivered to the structure would be 53.03 kip-inches (in which k = load/deflection =



F_x = FORCE ON VESSEL LATERALLY
F_y = FORCE ON VESSEL AXIALLY
$(KE)_x$ = INERTIA FORCE (KINETIC ENERGY) LATERALLY
$(KE)_y$ = INERTIA FORCE (KINETIC ENERGY) AXIALLY
F_r = RESULTANT FORCE ON PROTECTIVE SYSTEM
F_f = FRICTIONAL OR TANGENTIAL FORCE ON PROTECTIVE SYSTEM
F_n = NORMAL FORCE ON PROTECTIVE SYSTEM
θ = IMPACT ANGLE
C_g = CENTER OF GRAVITY

If the impact angle becomes 90°, this would simulate a head on collision

Fig. 1.63

TABLE 1-11

INDUCED MAXIMUM PILE STRESSES (KSI)

Bridge	$V = 1^{knot}$		$V = 3^{knots}$		$V = 5^{knots}$		Ultimate Stress
	W.O.F.	W.F.	W.O.F.	W.F.	W.O.F.	W.F.	(ksi)
$W = 1000^T$							
Napa River Bridge	8.82	1.55	38.23	4.77	75.76	7.92	5.00
Third Street Bridge	6.90	2.38	28.70	7.30	55.79	12.14	5.00
Steamboat Slough Bridge	56.13		280.48		553.26		5.00
Hoquiam River Bridge	.81	.055	3.89	.17	7.98	.29	5.00
Schuylkill River Bridge	.25	.02	1.22	.07	2.54	.11	5.00
Connecticut River Bridge	16.59	17.51	88.40	78.95	214.55	152.90	5.00
Mare Island Navy Yard		2.00		17.70		49.11	36.00
Swinomish Channel Bridge	25.82	27.61	77.47	81.84	163.42	160.44	5.00
$W = 10{,}000^T$							
Napa River Bridge	41.00	5.03	178.33	15.13	353.43	25.33	5.00
Third Street Bridge	30.75	7.70	128.11	23.15	250.75	38.65	5.00
Steamboat Slough Bridge	305.20		1500.68		3166.35		5.00
Hoquiam River Bridge	4.19	.18	19.50	.55	39.50	.90	5.00
Schuylkill River Bridge	1.32	.07	6.31	.20	13.00	.31	5.00
Connecticut River Bridge	99.78	84.61	535.93	305.45	1135.76	596.58	5.00
Mare Island Navy Yard		19.48		174.85		488.85	36.00
Swinomish Channel Bridge	81.66	86.30	391.96	357.90	1051.66	689.68	5.00
$W = 100{,}000^T$							
Napa River Bridge	191.29	15.96	830.28	48.23	1634.94	80.51	5.00
Third Street Bridge	137.26	24.40	585.65	73.45	1152.58	122.59	5.00
Steamboat Slough Bridge	1620.19		8019.62		16790.36		5.00
Hoquiam River Bridge	20.98	.58	94.99	1.54	190.45	2.68	5.00
Schuylkill River Bridge	6.80	.21	31.91	.62	64.86	1.09	5.00
Connecticut River Bridge	578.89	325.59	2890.82	1197.65	6100.05	2131.27	5.00
Mare Island Navy Yard		194.56		1731.00		4832.75	36.00
Swinomish Channel Bridge	493.24	382.48	2655.01		5625.17		5.00

TABLE 1-12

SUMMARY OF ENERGY DATA

$W = 1000^T$, $V = 1^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{FX}}{E_{NF}}$ (%)	Comment
Napa River Bridge	1000	1	1095.75	979.43	53.03	1032.46	10.57	O.K. with Fenders and Failed without Fenders
Third Street Bridge	1000	1	1083.25	947.62	107.10	1054.72	12.52	O.K. with Fenders and Failed without Fenders
Steamboat Slough Bridge	1000	1	1149.13					Failed with and without Fenders
Hoquiam River Bridge	1000	1	1115.63	972.31	60.77	1033.08	10.85	O.K. with and without Fenders
Schuylkill River Bridge	1000	1	1066.00	976.27	56.21	1032.48	8.42	O.K. with and without Fenders
Conneticut River Bridge	1000	1	1094.26	328.23	750.66	1078.89	70.00	Failed with and without Fenders
Mare Island Navy Yard	1000	1	1046.35	1046.35	17.44	1063.79	1.67	O.K. with Fenders Failed without Fenders
Swinomish Channel Bridge	1000	1	1094.19	265.02	815.19	1080.21	75.78	Failed with and without Fenders

TABLE 1-13

SUMMARY OF ENERGY DATA

$W = 1000^T$, $V = 3^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comment
Napa River Bridge	1000	3	1095.75	979.43	53.03	1032.46	10.57	O.K. with Fenders and Failed without Fenders
Third Street Bridge	1000	3	9932.50	9091.91	681.95	9773.86	8.46	Failed with and without Fenders
Steamboat Slough Bridge	1000	3	9906.40					Failed with and without Fenders
Hoquiam River Bridge	1000	3	10166.00	9376.01	290.97	9666.98	7.77	O.K. with and without Fenders
Schuylkill River Bridge	1000	3	9774.00	9445.85	267.62	9713.47	3.36	O.K. with and without Fenders
Conneticut River Bridge	1000	3	9694.50	4577.70	6023.62	10601.32	52.78	Failed with and without Fenders
Mare Island Navy Yard	1000	3	98.66	9866.89	164.11	10031.00	1.67	O.K. with Fenders and Failed without Fenders
Swinomish Channel Bridge	1000	3	9848.13	2620.20	7118.76	9738.96	73.39	Failed with and without Fenders

TABLE 1-14

SUMMARY OF ENERGY DATA

$W = 1000^T$, $V = 5^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comment
Napa River Bridge	1000	5	28217.00	25629.56	1309.95	26939.51	9.17	Failed with and without Fenders
Third Street Bridge	1000	5	27170.00	25208.53	1594.37	26802.90	7.22	Failed with and without Fenders
Steamboat Slough Bridge	1000	5	27461.55					Failed with and without Fenders
Hoquiam River Bridge	1000	5	28305.25	26177.95	939.48	27117.43	7.52	Failed with and without Fenders
Schuylkill River Bridge	1000	5	27666.00	25738.72	985.09	26723.81	6.97	O.K. with and without Fenders
Connecticut River Bridge	1000	5	27200.40	15117.82	15307.72	30425.54	44.42	Failed with and wihtout Fenders
Mare Island Navy Yard	1000	5	27622.61	27622.61	470.74	28093.35	1.70	Failed with and without Fenders
Swinomish Channel Bridge	1000	5	29399.35	9224.71	20474.71	29699.42	68.62	Failed with and without Fenders

TABLE 1-15

SUMMARY OF ENERGY DATA

$W = 10{,}000^{T}, V = 1^{K}$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comment
Napa River Bridge	10000	1	11245.50	10443.27	316.16	10759.43	7.13	O.K. with Fenders and Failed without Fenders
Third Street Bridge	10000	1	11384.00	10122.57	744.60	10867.17	11.08	Failed with and without Fenders
Steamboat Slough Bridge	10000	1	10567.80					Failed with and without Fenders
Hoquiam River Bridge	10000	1	11354.00	10430.88	324.75	10755.63	8.13	O.K. with and without Fenders
Schuylkill River Bridge	10000	1	11280.00	10521.72	290.48	10812.20	6.72	O.K. with and without Fenders
Connecticut River Bridge	10000	1	10926.50	5199.82	6632.97	11832.79	52.41	Failed with and without Fenders
Mare Island Navy Yard	10000	1	10977.03	10977.03	107.25	11164.28	1.71	O.K. with and without Fenders
Swinomish Channel Bridge	10000	1	10942.96	2912.54	7916.74	10829.28	73.38	Failed with and without Fenders

TABLE 1-16

SUMMARY OF ENERGY DATA

$W = 10{,}000^{T}$, $V = 3^{K}$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comment
Napa River Bridge	10000	3	97370.00	93006.78	5035.21	98041.99	4.48	Failed with and without Fenders
Third Street Bridge	10000	3	11384.00	10122.57	744.60	10867.17	11.08	Failed with and without Fenders
Steamboat Slough Bridge	10000	3	98112.00					Failed with and without Fenders
Hoquiam River Bridge	10000	3	99044.50	94458.24	3436.64	97894.88	4.63	O.K. with Fenders Failed without Fenders
Schuylkill River Bridge	10000	3	102812.50	9413.04	2850.32	96988.36	8.44	O.K. with Fenders Failed without Fenders
Connecticut River Bridge	10000	3	106379.00	57679.69	46970.95	104650.64	45.78	Failed with and without Fenders
Mare Island Navy Yard	10000	3	99561.63	99561.63	1691.63	101253.26	1.70	Failed with and without Fenders
Swinomish Channel Bridge	10000	3	103162.15	44340.97	68439.46	112780.42	57.02	Failed with and without Fenders

TABLE 1-17

SUMMARY OF ENERGY DATA

$W = 10{,}000^T$, $V = 5^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comment
Napa River Bridge	10000	5	274690.00	260775.61	12161.97	272937.58	5.07	Failed with and without Fenders
Third Street Bridge	10000	5	277078.00	256082.19	12274.37	268356.56	7.58	Failed with and wihtout Fenders
Steamboat Slough Bridge	10000	5	273401.00					Failed with and without Fenders
Hoquiam River Bridge	10000	5	272282.00	263171.78	9021.66	272193.44	3.35	O.K. with Fenders Failed without Fenders
Schuylkill River Bridge	10000	5	277770.00	264157.99	6537.90	270695.89	4.90	O.K. with Fenders Failed without Fenders
Connecticut River Bridge	10000	5	276725.00	185822.22	106541.16	292363.38	32.85	Failed with and without Fenders
Mare Island Navy Yard	10000	5	276535.01	276535.01	4716.49	281251.50	1.71	Failed with and without Fenders
Swinomish Channel Bridge	10000	5	296032.00	153478.18	170016.68	323494.86	48.15	Failed with and without Fenders

TABLE 1-18

SUMMARY OF ENERGY DATA

$W = 100{,}000^T,\ V = 1^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	$E_{Ff}+E_{FP}$ (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comments
Napa River Bridge	100000	1	108640.00	103452.35	5519.17	108971.52	4.78	Failed with and without Fenders
Third Street Bridge	100000	1	111538.50	102038.79	5406.13	1074444.92	8.52	Failed with and without Fenders
Steamboat Slough Bridge	100000	1	110804.69					Failed with and without Fenders
Hoquiam River Bridge	100000	1	108171.00	104873.13	3886.64	108759.77	3.05	O.K. with Fenders Failed without Fenders
Schuylkill River Bridge	100000	1	115290.00	104692.54	3046.74	107739.28	9.19	O.K. with Fenders Failed without Fenders
Connecticut River Bridge	100000	1	108583.75	64581.92	51368.85	115950.77	40.52	Failed with and without Fenders
Mare Island Navy Yard	100000	1	110620.32	110620.32	1884.04	112504.36	1.70	Failed with and without Fenders
Swinomish Channel Bridge	100000	1	107815.12	50225.56	74743.88	124969.44	53.42	Failed with and without Fenders

TABLE 1-19

SUMMARY OF ENERGY DATA

$W = 100,000^T$, $V = 3^K$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	E_{Ff+FP} (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comments
Napa River Bridge	100000	3	973200.00	945832.28	37627.70	983459.98	2.81	Failed with and without Fenders
Third Street Bridge	100000	3	1015145.00	924928.98	40100.57	965029.55	8.89	Failed with and without Fenders
Steamboat Slough Bridge	100000	3	946000.00					Failed with and without Fenders
Hoquiam River Bridge	100000	3	972920.00	940858.51	32560.86	973419.37	3.30	O.K. with Fenders Failed without Fenders
Schuylkill River Bridge	100000	3	1000890.00	956532.22	15610.68	972142.90	4.43	O.K. with Fenders Failed with Fenders
Connecticut River Bridge	100000	3	970612.50	722335.41	308194.08	1030529.49	25.58	Failed with and without Fenders
Mare Island Navy Yard	100000	3	995602.12	995602.12	16855.79	1012457.91	1.09	Failed with and without Fenders
Swinomish Channel Bridge	100000	3	1005941.00					Failed with and without Fenders

TABLE 1-20

SUMMARY OF ENERGY DATA

$W = 100{,}000^{T}, \ V = 5^{K}$

Place	Weight (T)	Velocity (Knots)	E_{NF} (k-in)	E_{Ff} (k-in)	E_{FP} (k-in)	E_{Ff+FP} (k-in)	$\frac{E_{NF}-E_{Ff}}{E_{NF}}$ (%)	Comments
Napa River Bridge	100000	5	2739600.00	2637092.20	95553.57	2732645.77	3.74	Failed with and without Fenders
Third Street Bridge	100000	5	2743950.00	2575510.70	104576.03	2680086.23	6.14	Failed with and without Fenders
Steamboat Slough Bridge	100000	5	2669745.00					Failed with and without Fenders
Hoquiam River Bridge	100000	5	2659000.00	2639168.10	73669.33	2712837.43	0.75	O.K. with Fenders Failed without Fenders
Schuylkill River Bridge	100000	5	2750280.00	2676943.60	32793.60	2709737.20	2.67	O.K. with Fenders Failed without Fenders
Connecticut River Bridge	100000	5	2769012.00	2086706.90	709340.09	2796046.99	24.64	Failed with and without Fenders
Mare Island Navy Yard	100000	5	2765292.30	2765292.30	47043.28	2812335.58	1.70	Failed with and without Fenders
Swinomish Channel Bridge	100000	5	2691406.20					Failed with and without Fenders

11.80 kips/inch) as per Figure 1-44. Thus, the structure absorbed 10.57 percent of the maximum possible load and the fender system absorbed the rest.

Now in order to determine if the structure (with or without fenders) has failed the induced maximum pile stresses (kips per square inch) need to be checked. In viewing Table 1-11 and looking at the Napa River Bridge with the above given parameters it is shown that the structure failed when unprotected but when protected survives the impact.

Thus, this bridge, for the above conditions, would need a fender system with an appropriate k = 11.80 kips/inch. By checking the various manufacturers catalogs an appropriate fender may be found.

In those bridges in which the structure failed, with and without fenders, it simply means that the protective system needs to be one in which the k value is smaller and then the system would be adequate. Thus, for the Napa River Bridge a fender with a k value of 11.80 kips/inch (weight of 1000 tons and a velocity of 1 knot) would not be adequate for a vessel weighing 10000 tons travelling at a velocity of 1 knot. The k value would have to be reduced to approximately 5 kips/inch.

When the term k is utilized it means the equation written from the k versus Δ curves (load/deflection verses deflection). The k value is equal to $a\Delta^2 + b\Delta + c$. All fender systems regardless of the type and/or material have a k value Thus, k as used here does not represent any particular fender system; but, represents andy fender system that has the appropriate k value.

CHAPTER II: Berwick Bay, Louisiana

Introduction

Of all the bridges analyzed, the bridges which cross the Atchafalaya River from Morgan City, La. to Berwick, La. have suffered the most casualties. In the period FY 1970 through FY 1974, there were fifty two (52) casualties. With few exceptions, causes of the casualties involved current or operator misjudgement of effects of current.

Three bridges cross Berwick Bay, two highway and one railroad. The first highway bridge was completed in November 1934 and is a fixed bridge with a horizontal clearance of 583 feet with a 50 feet vertical clearance at high water. The second highway bridge, located about 300 feet downstream has a horizontal span of 525 feet and a vertical clearance of 73 feet at high water.

Approximately 1200 feet downstream from the highway bridges is the Southern Pacific Railroad Bridge. The present railroad bridge was built in 1907. It was originally design as a swing bridge with a horizontal clearance of 108 feet. Because of numerous collisions, this swing span was replaced in April 1971, by a new lift span. The new bridge was authorized under the Truman Hobbs Act. It has a vertical lift span with a horizontal clearance of 320 feet and a vertical clearance in the raised position of 73 feet. This Southern Pacific Railroad Bridge is the bridge of concern in this publication.

The Waterway:

The bridge approach from the north is the most important since the great majority of accidents occurred to tows operating downstream. The passage from upriver is complicated by a sharp bend which begins at the intersection of three passages known as Stouts Pass, Drews Pass, and the Port Allen Route. The southern end of the bend terminates approximately 3/4 of a mile above the highway bridges. The course change required in the bend is approximately 90 degrees with a radius of one mile.

Tows rounding this bend tend to slide toward the Berwick side of the river unless they hug the bank on the Morgan City side. Coming out of the south end of the bend the stern of the tows is sometimes set toward the center of the river by the cross current. Care must be taken to counteract the higher current forces on the stern and prevent the tow from rotating and becoming out-of-shape. In many casualty situations tows that get out-of-shape in the bend do not have sufficient time to correct before drifting down on the bridge.

Passage thru the bridges must be made at an angle to the normal current direction in order to make the sharp turn in the river just below the railroad bridge. The normal passage thru these bridges is to pass the highway bridge between the center of the span and the right descending pier. The tow then drives for the right descending pier on the railroad bridge to counteract the cross current which tends to set the tow down on the left descending pier. The tow will pass as close to the right pier as practical in order to make the turn into the bend below.

The passage upriver is less hazardous but still complicated by the sharp turn just below the railroad bridge. In the bend the tow must cross the river and turn into the current to line-up for the passage thru the railroad bridge. At times of extreme high tide the current flow is reversed and flows north thru the bridges. The operator must be aware of this condition and plan his approach accordingly.

The primary problem with this waterway is the bend above the bridge and the current. The main causes of casualties are strong currents, out-of-shape, and wind and currents.

Method of Approach

The method of approach utilized in the analysis of the Southern Pacific Railroad Bridge was to conduct a hydraulic laboratory mdodel test and measure the appropriate parameters to determine the adequacy of the bridge protective system and/or device.

The hydraulic models were constructed of concrete on a wave-basin floor inside a small building (to protect them from wind and rain). Breakwaters were constructed of crushed stone. Waves were produced to scale by a movable, plunger-type wave machine which rotates about the vertical axis such that it can change the direction of the wave at various points in the channel as it is unique to Berwick Bay area. Wave filters were utilized with the wave machine to reflect waves and serve the same function as absorbing beaches. Wave heights were measured with electrical wave-height gages. The hydraulic model was constructed to a linear scale of 1:150 (model to prototype). The models were designed and operated in accordance with Froude's model laws and the following Model to Prototype Relationships were used when necessary.

As for the vessel used for impact as well as the bridge protective systems and/or devices the same scales as above were utilized. The vessel was attached to a ball pinned connection which allowed free rotation of the vessel. A motorized driving machine was calibrated and utilized to obtain desired speeds and directions of approach. The mass of the vessel was also scaled as above.

Model: Prototype Relationships

Characeteristics	Dimensions	Model: Prototype Scales
Length	L	$L_r = 1:150$
Area	L^2	$A_r = L_r^2 = 1:22,500$
Volume	L^3	$\overline{V}_r = L_r^3 = 1:3,375,000$
Time	T	$T_r = L_r^{1/2} = 1:12.25$
Velocity	L/T	$V_r = L_r^{1/2} = 1:12.25$
Unit Pressure	F/L^2	$P_r = L_r \gamma_r = 1:150$
Force	F	$F_r = L_r^3 \gamma_r = 1:3,375,000$
Weight	F	$W_r = L_r^3 \gamma_r = 1:3,375,000$

Modeling Assumptions

In the hydraulic analysis of the Southern Pacific Railroad Bridge at Morgan City, La. the analysis was performed with and without fenders (bridge protective systems and devices). The bridge protective systems and/or devices of the Southern Pacific Railroad Bridge were tested in much the same manner as those bridges tested using the computer program (the difference being a hydraulic analysis rather than a computer analysis) under the following criteria:

1. Various sizes of vessels now using the waterways;
2. Various velocities and direction of approach (of the worst case).
3. Various winds (as incorporated into the velocity)
4. Various tidal currents (as incorporated into the velocity)
5. Existing soil characteristics
6. The rigidity and energy absorbing characteristic of the bridge and pier protective systems and/or vessels (as measured).

The performance characteristics of the system were measured in terms of:

1. Velocity of vessel at every instance of impact;
2. Load deformation of the protective system at every instance of impact;
3. Energy absorbed by the protective system and the vessels at every instance of impact.

Southern Pacific Railroad Bridge

In the analysis of the Southern Pacific Railroad Bridge at Morgan City, La. the velocities chosen (at the time of impact) were 1, 3, and 5 knots. This particular bridge also had four different types of bridge and/or pier protective systems and/or devices as follows:

1. Continuous Round Pile System
2. Steel Shut Pile
3. Skewed Cap
4. A General Fender System

Each of these systems were modeled and analyzed by the hydraulic analysis method. Figure 2.1 to 2.4 illustrate the structural details of each type of bridge protective system. Tables 2.1 to 2.4 illustrate the modeling techniques employed (material properties, soil properties, pile data, and system data) necessary for the proper hydraulic modeling. (These figures and tables are shown in Appendix D).

Results

The results of the hydraulic analysis are shown in Tables 2.5 to 2.8 for the maximum induced stress acquired and Tables 2.9 to 2.12 for the energies absorbed.

First viewing Table 2.5: Continuous Round Pile System the hydraulic analysis was performed with and without fenders. With fenders the continuous round pile system was adequate as a bridge and/or pier protective system up to and including a vessel weighing 100000 tons and traveling at 1 knot. However, beyond that point the protective system failed due to a plastic reaction of the soil. Without fenders the continuos round pile system was adequate up to an including a vessel weighing 10000 tons traveling at 1 knot. Beyond this point the bridge would fail.

Table 2.6: Steel Sheet Pile data indicates that as a bridge and/or pier protective system the sheet pile as designed and constructed is inadequate for any vessel at any speed. In other-words a small vessel weighing 1000 tons traveling at 1 knot would destroy the system whether the steel Sheet Pile system was with or without fenders.

Table 2.7: Skewed Shape Cap System seems to be adequate with fenders up to and including a vessel weighing 10,000 tons traveling at 3 knots and without fenders up to an including a vessel weighing 1000 tons traveling at 3 knots.

The final bridge and/or pier protective system is the fendering system with data as shown in Table 2.8. With or without fenders this system is inadequate as a protective system for this bridge.

Tables 2.9 to 2.12 where appropriate; that is, where a bridge protective system was found to be adequate gives you the amount of energy remaining in the protective system.

Discussion of the Results

In utilizing the summary of energy data tables; only the Continuous

Round Pile bridge protective system will be discussed. After a discussion is presented it should be obvious to the reader the theory behind the results of the remaining bridge protective systems and/or devices.

Table 2.9 shows the effects of a head on vessel collision with a fixed structure (with and without fenders) as reported by the hydraulic analysis. In the very first case the vessel is traveling at a velocity of 1 knot and weighs 1000 tons at the time of impact. In viewing the table the energy delivered to the unprotected structure would be 1191 kip inches. However, if protected with continuous round piles the load delivered to the structure would be 104.8 kip inches. Thus, the structure absorbed 8.55 percent of the maximum load and the continuous round pile system absorbed the rest along with the vessel.

Now in order to determine if the structure (with or without fenders) has failed the induced maximum pile stresses (kips per square inch) need to be checked. In viewing Table 2.5 and viewing the same parameters as above it is shown that the system did not fail with or without fenders (for the above parameters).

In those cases where the bridge protective system and/or structure failed (with or without fenders), it simply means that the protective system needs to be one in which the k value (load/deflection) is smaller and the system would be adequate.

For example in viewing Table 2.5 Continuous Round Pile System it is noted that for a vessel weighing 100000 tons and traveling at 3 knots this protective system would be inadequate. The k value of this system as determined by Fig. 2.7 Continuous Round Pile System is approximately 190 kips/inch.

When the term k is utilized it means the equation written from the k versus Δ curves (load/deflection versus deflection). The k value is equal to $a\Delta^2 + b\Delta + c$.

All fender systems regardless of the type and/or material have a k value. Thus, k as used here does not represent any particular fender system; but, represent any fender system that has the appropriate k value. Table 2.13 shows a comparison of the computer results to the hydraulic results.

Comments on Berwick Bay

The primary problem with this waterway is the high velocity current which causes difficulty in the tows lining up correctly for the bridges. These tows must be in correct alignment for the highway bridges as well as the railroad bridge because of the restricted maneuvering room between the structures. These problems coupled with the poor soil conditions make it almost impossible to provide adequate protection for the Southern Pacific Transportation Railroad Bridge. The soil is of poor quality and highly plastic. Any structure (dolphin, cell, piles, etc.) placed in the waterway will only be forced to topple over or slide towards the bridge upon vessel impact. The system(s) will absorb some of the energy but damage to the bridge may still result. A computer analysis as well as a hydraulic analysis of the four main protective systems show that they are all basically inadequate due to the poor soil conditions although structurally very adequate.

One possible solution might be to provide a gravity type fendering system with rubber mounted on the face of the massive concrete structure hung from the bridge (provided that the bridge can take the necessary load). This system would have to be designed in such a way that the energy due to the force of impact is not transferred to the substructure of the bridge.

Another approach would be the use of a floating buoy structure which would extend around the pier and act to catch the vessels before they strike the bridge. Special consideration would have to be given to the design of these floating buoys and their attachment to the river system.

There are probably other systems and/or devices which should be studied such as warning systems and navigational aids.

The only safe system is to relocate the railroad bridge between the two highway bridges. In this case the vessels will be lining up for one bridge structure rather than two. There will exist wider clearances and more room to maneuver on each side of the structure eventually enabling vessels to line up for the bend south of the railroad bridge as they immediately pass the bridge structures.

TABLE 2.5: Continuous Round Pile System

BERWICK BAY ROUND SHAPE CAP PILES

Induced Max. Pile Stress

Place	Ship Data		Without Fenders					With Fenders				
	Weight (t)	Velocity (knots)	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry
Berwick	1000	1	76.23	29266	6.13	36.30	O.K.	5.30	7787	1.63	36.30	O.K.
Bay	1000	3	76.23	87803	18.38	36.30	O.K.	5.30	23361	4.89	36.30	O.K.
	1000	5	76.23	148963	31.19	36.30	O.K.	5.30	38936	8.15	36.30	O.K.
	10000	1	76.23	92546	19.38	36.30	O.K.	5.30	24624	5.16	36.30	O.K.
	10000	3	-	-	-	36.30	Failed*	5.30	73872	15.47	36.30	O.K.
	10000	5	-	-	-	36.30	Failed*	5.30	123444	25.85	36.30	O.K.
	100000	1	-	-	-	36.30	Failed*	5.30	77869	16,30	36.30	O.K.
	100000	3	-	-	-	36.30	Failed*	-	-	-	36.30	Failed*
	100000	5	-	-	-	36.30	Failed*	-	-	-	36.30	Failed*

*: The reason of failure is that a plastic reaction has been obtained from the soil surrounding the pile at every point

TABLE 2.6: Steel Sheet

BERWICK BAY STEEL SHEET PILE

Induced Max. Pile Stress:

Place	Ship Weight (t)	Data Velocity (knots)	Without Fenders K (k/in)	M (k-in)	σ (ksi)	σ_u (ksi)	Commentry	With Fenders K (k/in)	M (k-in)	σ_c (ksi)	σ_u (ksi)	Commentry
	1000	1	-	-	-	0.4	Failed*	-	-	-	0.4	Failed*
Berwick	"	3	-	-	-	"	" *	-	-	-	-	" *
	"	5	-	-	-	"	" *	-	-	-	-	" *
Bay	10000	1	-	-	-	"	" *	-	-	-	-	" *
	"	3	-	-	-	"	" *	-	-	-	-	" *
	"	5	-	-	-	"	" *	-	-	-	-	" *
	100000	1	-	-	-	"	" *	-	-	-	-	" *
	"	3	-	-	-	"	" *	-	-	-	-	" *
	"	5	-	-	-	"	" *	-	-	-	-	" *

*: The reason of failure is that a plastic reaction has been obtained from the soil surrounding the pile at every point.

TABLE 2.7: Skewed Shape

BERWICK BAY SKEW SHAPE CAP PILE

Induced Max. Pile Stress

Place	Ship Data		Without Fenders					With Fenders				
	Weight (t)	Velocity (knots)	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry
Berwick	1000	1	138.83	56463	11.82	36.30	O.K.	5.48	11217	2.35	36.30	O.K.
Bay	1000	3	138.83	171790	35.97	36.30	O.K.	5.48	33651	7.05	36.30	O.K.
	1000	5	-	-	-	36.30	Failed*	5.48	56085	11.74	36.30	O.K.
	10000	1	138.83	181975	38.10	36.30	Failed*	5.48	35471	7.43	36.30	O.K.
	10000	3	-	-	-	36.30	Failed*	5.48	106417	22.28	36.30	O.K.
	10000	5	-	-	-	36.30	Failed*	5.48	180646	37.82	36.30	Failed*
	100000	1	-	-	-	36.30	Failed*	5.48	112174	23.49	36.30	O.K.
	100000	3	-	-	-	36.30	Failed*	-	-	-	36.30	Failed*
	100000	5	-	-	-	36.30	Failed*	-	-	-	36.30	Failed*

*: The reason of failure is that a plastic reaction has been obtained from the soil surrounding the pile at every point.

TABLE 2.8: Fender System

BERWICK BAY CONTINOUS PILE GROUPS

Induced Max. Pile Stress (ksi)

Place	Ship Data		Without Fenders					With Fenders				
	Weight (t)	Velocity (knots)	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry	K (k/in)	M (k-ft)	σ (ksi)	σ_u (ksi)	Commentry
Berwick Bay	1000	1	14.31	8407.2	206.67	26.00	Failed	7.84	2669.7	65.63	36.00	Failed
	"	3	27.14	40427.3	993.77	36.00	"	8.51	8813.7	216.66	36.00	"
	"	5	37.21	83363.1	2049.21	36.00	"	8.83	15053.1	370.03	36.00	"
	10000	1	28.02	43575.8	1071.17	36.00	"	8.59	9316.8	229.02	36.00	"
	"	3	55.44	204109.3	5017.37	36.00	"	9.35	31128.5	765.19	36.00	"
	"	5	76.70	413644.6	10168.12	36.00	"	9.22	49716.5	1222.12	36.00	"
	100000	1	57.32	219597.2	5398.09	36.00	"	8.85	30612.2	752.5	36.00	"
	"	3	115.74	997398.1	24517.81	36.00	"	9.37	96062.1	2361.38	36.00	"
	"	5	160.54	1999760.9	49157.66	36.00	"	9.45	161882.7	3979.36	36.00	"

TABLE 2.9: Continuous Round Pile

Energies:

Place	Ship Weight (t)	Ship Velocity (knots)	ENF (K-in)	EFf (k-in)	EFp (k-in)	EFf + EFp (k-in)	$\frac{(ENF - EFf)}{ENF}$ %
	1000	1	1191.0	1089.2	104.8	1194.0	8.55
Berwick	"	3	10714.0	9802.6	907.4	10710.0	8.51
Bay	"	5	29760.0	27229.5	2576.5	29806.0	8.50
	10000	1	11905.0	10891.6	1008.4	11900.0	8.51
	"	3	-	98028.4	9073.6	107102.0	-
	"	5	-	272300.0	25206.0	297506.0	-
	100000	1	-	108920.0	10083.0	119003.0	-
	"	3	-	-	-	-	-
	"	5	-	-	-	-	-

TABLE 2.10: Steel Sheet Pile

Energies:

Place	Ship Weight (t)	Ship Velocity (knots)	ENF (k-in)	EFf (k-in)	EFp (k-in)	EFf + EFp (k-in)	$\frac{ENF - EFf}{ENF}$ (%)
Berwick	1000	1	-	-	-	-	-
Bay	"	3	-	-	-	-	-
	"	5	-	-	-	-	-
	10000	1	-	-	-	-	-
	"	3	-	-	-	-	-
	"	5	-	-	-	-	-
	100000	1	-	-	-	-	-
	"	3	-	-	-	-	-
	"	5	-	-	-	-	-

TABLE 2.11: Skewed Cap

Energies:

Place	Ship Weight (t)	Ship Velocity (knots)	ENF (k-in)	EFf (k-in)	EFp (k-in)	EFf + EFp (k-in)	$\frac{(ENF - EFf)}{ENF}$ %
Berwick Bay	1000	1	1192.0	1145.4	48.6	1194.0	3.91
	"	3	10726.0	10309.3	427.7	10737.0	3.88
	"	5	-	28636.2	1183.8	29820.0	-
	10000	1	11918.0	11454.5	475.5	11930.0	3.89
	"	3	-	103090.3	4248.7	107339.0	-
	"	5	-	286361.9	11784.1	298146.0	-
	100000	1	-	114544.6	4719.4	119264.0	-
	"	3	-	-	-	-	-
	"	5	-	-	-	-	-

TABLE 2.12: FENDER

Energies:

Place	Ship Weight (t)	Ship Velocity (knots)	ENF (k-in)	EFf (k-in)	EFp (k-in)	EFf + EFp (k-in)	$\frac{(ENF - EFf)}{ENf}$ %
	1000	1	1087.1	863.9	273.8	1137.7	20.53
Berwick	"	3	9803.1	8308.5	1562.6	9871.1	15.25
Bay	"	5	27443.6	24010.4	3525.2	27535.6	12.51
	10000	1	10902.0	9271.2	1702.3	10973.5	14.96
	"	3	99080.7	94970.6	7316.1	102286.7	4.15
	"	5	275479.5	249790.8	22578.9	272369.7	9.33
	100000	1	110108.4	95864.2	13480.2	109844.4	12.94
	"	3	991812.5	910290.2	64367.6	974657.8	8.22
	"	5	2754109.3	2546676.6	150653.1	2697329.7	7.53

TABLE 2.13: Comparison of Hydraulic Modeling vs. Computer Modeling

Structure	Vessel Data		Without Fenders		With Fenders		σ_u (ksi)	Commentary	
	Weight (t)	Velocity (knots)	Modeling σ(ksi)	Computer σ(ksi)	Modeling σ(ksi)	Computer σ(ksi)		Without	With
Round Shape Cap Pile	1000	1	6.13	5.51	1.63	1.61	36.30	ok	ok
	1000	3	18.38	17.19	4.89	4.58	36.30	ok	ok
	1000	5	31.19	29.15	8.15	7.72	36.30	ok	ok
	10000	1	19.38	18.49	5.16	4.77	36.30	ok	ok
	10000	3	--	36.85	15.47	14.65	36.30	Failed	ok
	10000	5	--	44.61	25.85	24.79	36.30	Failed	ok
	100000	1	--	42.33	16.30	15.59	36.30	Failed	ok
	100000	3	--	48.94	--	37.01	36.30	Failed	Failed
	100000	5	--	60.66	--	44.67	36.30	Failed	Failed
Steel Sheet Pile	1000	1	--	0.45	--	0.41	0.40	Failed	Failed
	1000	3	--	0.54	--	0.48	0.40	Failed	Failed
	1000	5	--	0.67	--	0.59	0.40	Failed	Failed
	10000	1	--	--	--	--	0.40	Failed	Failed
	10000	3	--	--	--	--	0.40	Failed	Failed
	10000	5	--	--	--	--	0.40	Failed	Failed
	100000	1	--	--	--	--	0.40	Failed	Failed
	100000	3	--	--	--	--	0.40	Failed	Failed
	100000	5	--	--	--	--	0.40	Failed	Failed

TABLE 2:13: Comparison of Hydraulic Modeling vs. Computer Modeling (Cont'd.)

Structure	Vessel Data		Without Fenders		With Fenders		σ_u (ksi)	Commentary	
	Weight (t)	Velocity (knots)	Modeling σ (ksi)	Computer σ (ksi)	Modeling σ (ksi)	Computer σ (ksi)		Without	With
Skew Shape Cap Pile	1000	1	11.82	11.79	2.35	2.21	36.30	ok	ok
	1000	3	35.97	34.13	7.05	6.93	36.30	ok	ok
	1000	5	--	38.10	11.74	11.14	36.30	Failed	ok
	10000	1	--	37.52	7.43	7.03	36.30	Failed	ok
	10000	3	--	41.20	22.28	21.73	36.30	Failed	ok
	10000	5	--	44.61	--	37.82	36.30	Failed	Failed
	100000	1	--	43.59	23.49	36.79	36.30	Failed	ok
	100000	3	--	50.11	--	42.33	36.30	Failed	Failed
	100000	5	--	56.17	--	49.10	36.30	Failed	Failed
Continuous Pile Groups	1000	1	--	207	--	66	36	Failed	Failed
	1000	3	--	994	--	8813	36	Failed	Failed
	1000	5	--	2050	--	15053	36	Failed	Failed
	10000	1	--	1071	--	9317	36	Failed	Failed
	10000	3	--	5017	--	31128	36	Failed	Failed
	10000	5	--	10168	--	49716	36	Failed	Failed
	100000	1	--	5398	--	30612	36	Failed	Failed
	100000	3	--	24518	--	96062	36	Failed	Failed
	100000	5	--	49160	--	161883	36	Failed	Failed

Chapter III: Benjamin Harrison Memorial Bridge

Benjamin Harrison Memorial Bridge

Introduction

On 24 February 1977, the SS Marine Floridian, a bulk sulphur carrier, operated by Marine Transport Lines, Inc., was downbound on the James River about 2 miles below Hopewell, Virginia. About 500 yards from the Benjamin Harrison highway bridge, the steering system malfunctioned and the vessel veered to the left (north) of the channel and the raised center span of the bridge. The vessel collided with the support pier (which was unprotected by fenders, dolphins, cells, and/or platforms) between the bridge's northern approach causeway and its northern tower span and continued under the span until the vessel's starboard bridge wing struck the span. The northern end of the span then dropped across the main deck just forward of the aft-located deckhouse. The Marine Floridian was maintained in that position until 6 March 1977, when the span, including the northern main tower of the bridge, collapsed onto the vessel and into the river.

The Marine Floridian was moderately damaged and the bridge was extensively damaged. Total property damage was estimated to be $8,500,000. No one was injured except the bridge tender, who was injured slightly in evacuating the bridge.

The National Transportation Safety Board determined that the probable cause of the accident was inadequate maintenance and inspection of a manual transfer switch in the electrical circuit which opened by the force of gravity and thus interrupted electric power to the steering motor when the vessel was in a position from which it could not be stopped or steering gear power restored before it collided with the bridge.

Contributing to the cause of the collision was the operation of the vessel at a speed higher than necessary for a safe passage through the bridge opening, failure of the steering alarm to function, and the absence of a person on watch in the steering engine room which contributes to the delay in activating the alternate steering engine.

The main cause for the bridge collapse was inadequate protection of all bridge piers in the realm of possible collision with/by a vessel. Thus, one recommendation of the National Transportation Safety Board (NTSB) to the Federal Highway Administration was as follows:

> "Work with the U.S. Coast Guard to develop specifications for the design of dolphins, fenders, and other energy absorption and/or vessel redirection devices for the protection of both bridge and vessel during an accidental impact. Issue these design specifications along with guidelines and requirements for the placement of dolphins, fenders, and energy absorption and redirection devices."

This report entitled the Benjamin Harrison Memorial Bridge is the third volume of a study initiated (Laboratory Model Testing of Bridge Protective Systems and Devices) by the U.S. Coast Guard in the hope of satisfying the above recommendations. It represents the laboratory study conducted to determine the adequacy of the bridge protective system of the Benjamin Harrison Memorial Bridge at Hopewell, Virginia.

History of the Failure

At 1:00 P.M. on 23 February 1977, the tankership SS Marine Floridian, a bulk sulphur carrier owned by Marine Navigation Sulphur Carriers, Inc., and operated by Marine Transport Lines, Inc., arrived at the Allied Chemical Corporation dock on the James River at Hopewell, Virginia. The vessel had sailed from Beaumont, Texas, with a cargo of molten sulphur on 16 February 1977. Part of the cargo had been discharged at an intermediate stop at Morehead City, North Carolina.

Early on 24 February 1977, the discharge of the remaining cargo was completed at Hopewell and the vessel was ballasted for the return voyage. A pilot and a tugboat arrived at 5:40 A.M. to assist the vessel. Departure was scheduled for 6:30 A.M. to take advantage of the high slack tide, and to comply with the Coast Guard Notice to Mariners to transit the James River only during daylight hours. That notice also prescribed a maximum vessel draft of 20 feet and was promulgated as a precaution since ice had displaced some floating aids to navigation during the previous weeks.

Presailing tests were made on the vessel and no discrepancies were noted. The port steering engine operated properly both by the telemotor hydraulic control and by the electric control systems during the tests. Neither the starboard steering engine nor the steering alarm was tested. The steering was then selected for telemotor control from the helm. The port electric steering gear motor was being powered by the port electric feeder cables.

At 6:30 A.M., the tugboat assisted in moving the Marine Floridian from the dock and in turning it to head downriver in the channel. At 6:35 A.M., the vessel was headed downstream, the tug was released, and the engine was ordered half ahead. The pilot navigated the vessel from the wheelhouse under the normal supervisory relationship by the master. The appropriate positions were manned on the bow, and in the wheelhouse and the engine room. The steering engine was not manned nor was it required to be.

As the vessel proceeded downriver at half speed in the channel, the pilot prepared for passage in the channel under the center span of the Benjamin Harrison Memorial highway lift bridge. At 6:48 A.M., he requested via VHF radio channel 13 that the bridge tender raise the span. At 6:51 A.M., speed was increased to full ahead. At 6:52 A.M., the bridge tender sounded the prescribed signal to advise the pilot that the span had been raised to its open position. Also, lights on the center of the span changed aspect from red to green automatically when the span reached the raised position.

At 6:54 A.M., the pilot ordered right rudder to make the final turn from a course of about 056° true (T) to a course of about 071° T for passage through the bridge opening. The master, the pilot, the third mate, and the helmsman all noted that the ship's head and the rudder were not responding to the right turn order. The rudder angle indicator remained at 10° left as the helmsman put the wheel more than 10° right, and then hard right at the pilot's orders. The master and the pilot both testified that the estimated speed of the vessel over the ground at that time was between 6 and 7 knots, and that the vessel was about 500 yards from the bridge.

At 6:55 A.M., the pilot ordered the engine "back full" and the starboard anchor to be dropped. He then ordered "emergency back full". The throttleman responded promptly to the backing orders and the propeller was turning at 93 rpm's astern about ten (10) seconds after it was reversed.

The third mate used the sound-powered telephone to transmit the anchor order to the able seamen on watch on the bow. The starboard anchor was dropped promptly and it payed out properly. After the brake was applied, the anchor grabbed and fetched up intermittently and the chain jumped over the top of the wildcat when the forces of the fetching up were extreme. Immediately after the anchor was dropped, the general alarm was sounded and the danger signal was sounded on the ship's whistle.

Soon after the engine was reversed, the master called the engine room to report that the steering had failed. The master and the pilot continued to sound the ship's whistle to warn persons on the bridge. The bridge tender also sounded the bridge horn repeatedly. The vessel turned slowly to the left (north) of the channel and the raised center span of the bridge.

The vessel was slowed gradually but continued to move until it began to pass under the 241 foot long northern tower span of the bridge. At 6:56 A.M., the port bow collided with and demolished the downriver leg of the support pier under the joint between the northern tower span and the adjoining 112 foot long section of the bridge's approach causeway. The causeway section of steel-reinforcement (reinforced concrete) was displaced and fell, with two unoccupied highway vehicles, into the river on the vessel's port side in about 23 feet of water. The vessel continued under the tower span and the two kingposts located between the vessel's No. 1 and No. 2 cargo tanks were broken off at the main deck level when they contacted the span and fell aft and to port at an angle of about 45°. As the vessel came to rest, the outboard corner of the starboard wing of the ship's bridge struck the tower span; the span buckled slightly and its northern end dropped across the main deck just forward of the aft located deckhouse. The electric clock in the bridge tender's room stopped at 6:57 A.M. when electric power was interruped.

No major injuries were reported of any person. The vessel damage was estimated at $1,500,000 and the highway bridge damage was estimated at $7,000,000.

The National Transportation Safety Board determined that the probable cause of the accident was inadequate maintenance and inspection of a manual transfer switch in the electrical circuit which opened by the force of gravity and thus interrupted electric power to the steering motor when the vessel was in a position from which it could not be stopped or steering gear power restored before it collided with the bridge.

Contributing to the cause of the collision was the operation of the vessel at a speed higher than necessary for a safe passage through the bridge opening, failure of the steering alarm to function, and the absence of a person on watch in the steering engine room which contributed to the delay in activating the alternate steering engine.

The main cause for the bridge collapse was that it was hit by a vessel out of control and inadequate protection of all bridge piers in the realm of possible collison by a vessel. Thus, one major recommendation of the National Transportation Safety Board (NTSB) to the Federal Highway Administration was as follows:

> "Work with the U.S. Coast Guard to develop specifications for the design of dolphins, fenders, and other energy absorption and/or vessel redirection devices for the protection of both bridge and vessel during an accidental impact. Issue these design specifications along with guidelines and requirements for the placement of dolphins, fenders, and energy absorption and redirection devices."

Method of Approach

The method of approach utilized in the analysis of the Benjamin Harrison Memorial Bridge was to conduct a hydraulic laboratory model test and measure the appropriate parameters to determine the adequacy of the bridge protective system and/or device.

The hydraulic models were constructed of concrete on a wave-basin floor inside a small building (to protect them from wind and rain). Breakwaters were constructed of crushed stone. Waves were produced to scale by a movable, plunger-type wave machine which rotates about the vertical axis such that it can change the direction of the wave at various points in the channel. Wave filters were utilized with the wave machine to reflect waves and serve the same function as absorbing beaches. Wave heights were measured with electrical wave-height gages. The hydraulic model was constructed to a linear scale of 1:150 (model to prototype). The models were designed and operated in accordance with Froude's model laws and the following Model: Prototype Relationships were used when necessary.

Model: Prototype Relationships

Characteristics	Dimensions	Model: Prototype Scales
Length	L	$L_r = 1:150$
Area	L^2	$A_r = L_r^2 = 1:22{,}500$
Volume	L^3	$V_r = L_r^3 = 1:3{,}375{,}000$
Time	T	$T_r = L_r^{1/2} = 1:12.25$
Velocity	L/T	$V_r = L_r^{1/2} = 1:12.25$
Unit Pressure	F/L^2	$P_r = L_r \gamma_r = 1:150$
Force	F	$F_r = L_r^3 \gamma_r = 1:3{,}375{,}000$
Weight	F	$W_r = L_r^3 \gamma_r = 1:3{,}375{,}000$

As for the vessel used for impact as well as the bridge protective systems and/or devices the same scales as above were utilized. The vessel was attached to a ball pinned connection which allowed free rotation of the vessel. A motorized driving machine was calibrated and utilized to obtain desired speeds and directions of approach. The mass of the vessel was also scaled as above.

Modeling Assumptions

In the hydraulic analysis of the Benjamin Harrison Memorial Bridge at Hopewell, Virginia, the analysis was performed with and without fenders. The fendering system and/or piers of the Benjamin Harrison Memorial Bridge were tested in much the same manner as those bridges tested using the computer program under the following criteria:

1. Various sizes of vessels now using the waterways;
2. Various velocities and direction of approach (of the worst case);
3. Various winds (as incorporated into the velocity);
4. Various tidal currents (as incorporated into the velocity);
5. Existing soil characteristics;
6. The rigidity and energy absorbing characteristics of the fendering system and the vessel (as measured).

The performance characteristics of the system were measured in terms of:

1. Velocity of vessel at every instance of impact;
2. Load deformation of the protective system at every instance of impact;
3. Energy absorbed by the protective system and the vessels at every instance of impact.

In the analysis of the Benjamin Harrison Memorial Bridge, the velocities chosen (at the time of impact) were 1, 3, and 5 knots. These velocities were measured and assumed to take into account wind and tidal currents. The weights of the vessels were chosen as 1000, 10000, and 100000 tons. These weights were in the ralm of the majority of vessels using the waterway.

In addition to the above material properties, pile data, and system data were modeled as best as could be (Table 3.1 Benjamin Harrison Memorial Bridge Modeling Data). Many of the parameters given in Table 3.1 were obtained from Figures 3.1a and 3.1b; Benjamin Harrison Memorial Bridge Structural Details and Benjamin Harrison Memorial Bridge Modeling Details respectively. See Appendix E for Table 3.1 and Fig. 3.1 and 3.2.

Results

The results of the hydraulic analysis of the Benjamin Harrison Memorial Bridge are shown in Tables 3.2 and 3.3; Benjamin Harrison Memorial Bridge Summary of Stresses and Benjamin Harrison Memorial Bridge Summary of Energy respectively.

Discussion of the Results

In utilizing the summary of stresses and summary of energy tables, it becomes apparent that the existing fender system attached to the Benjamin Harrison Bridge is inadequate even for small vehicles (1000 tons) travelling at small velocities (1 knot).

Figure 3.2 Load vs. Deflection curves indicate the curves produced in any given situation with and without fenders. In order to make these fendering systems adequate; k versus Δ curves (load/deflection versus deflection) in which $k = a\Delta^2 + b\Delta + c$; the k value would have to be greatly reduced.

HOPEWELL BRIDGE

Induced Max. Pile Stress												
	Ship Data		Without Fenders					With Fenders				
Place	Weight (t)	Velocity (knots)	K (k/in)	M (k-ft)	(ksi)	(ksi)	Commentry	K (k/in)	M (k-ft)	(ksi)	(ksi)	Commentry
Hopewell	1000	1	9.61	631.2	44.6	5.00	Failed	21.60	811.6	57.4	5.00	Failed
"	"	3	17.37	3382.2	239.2	"	Failed	31.68	3467.0	745.2	"	Failed
"	"	5	22.95	7139.8	505.0	"	Failed	33.84	5535.2	391.5	"	Failed
"	10000	1	17.91	3658.4	258.7	"	Failed	32.07	3688.2	260.9	"	Failed
"	"	3	32.47	16980.3	1201.0	"	Failed	30.1	9275.2	656.0	"	Failed
"	"	5	43.0	38144.0	2697.8	"	Failed	32.29	17337.8	1226.2	"	Failed
"	100000	1	33.41	18333.5	1296.7	"	Failed	30.53	10015.9	708.4	"	Failed
"	"	3	61.57	97050.1	6864.1	"	Failed	35.40	37556.2	2656.2	"	Failed
"	"	5	81.92	203598.5	14400.0	"	Failed	37.83	67847.5	4586.5	"	Failed

TABLE 3.2: Summary of Stresses

Energies:

Place	Ship Weight (t)	Ship Velocity (knots)	ENF (k-in)	EFf (k-in)	EF_p (k-in)	$EFf + EF_p$	$\frac{(ENF - EFf)}{ENF}$ %
Hopwell Bridge	1000	1	1263.50	168.96	1105.25	1274.21	86.63
	1000	3	9880.00	2646.00	8376.91	11022.91	73.22
	1000	5	27008.00	13260.00	16317.63	29577.63	50.90
	10000	1	10829.50	3070.78	7839.72	10910.50	71.64
	10000	3	97607.10	61843.50	38776.00	100619.50	36.64
	10000	5	273050.00	182700.00	9614.14	279114.14	33.09
	100000	1	108242.40	68782.00	42644.32	111426.32	36.46
	100000	3	993377.00	742395.00	273131.96	1015526.96	25.27
	100000	5	2790150.00	2217565.20	615664.98	2833230.18	20.52

TABLE 3.3: Summary of Energy

CHAPTER IV: AREA STANDARDS

GENERAL

Scope

These specifications cover the design, construction, maintenance, and inspection of protective systems for railway bridge piers located in or adjacent to channels of navigable waterways.

Purpose

The purpose of the protective systems is to protect supporting piers of railway bridges from damage caused by accidental collision from floating vessels. Such protection should be designed to eliminate or reduce the impact energy transitted to the pier from the vessel, either by redirection of the force or by absorption, or dissipation of the energy, to non-destructive levels.

SPECIAL CONSIDERATIONS

Vessel

The size and type of vessel to be chosen as a basis for design of the pier protection should reflect the maximum vessel tonnage and velocity reasonably to be expected for the specific facility involved. Such tonnage are given in Tables 1-5 for various types of vessels.

Waterway

Consideration should be given to the exposure of the structure in the waterway, including the alignment of the channel, visibility

TABLE 1: INLAND WATERWAYS VESSELS

Designation	Horsepower	Carrying Capacity (kips)	Length (ft)	Beam (ft)	Draft (ft)
Towboats	1000-2000	-	117	30	7.6
	2000-4000	-	142	34	8.0
	4000-6000	-	160	40	8.6
Tugboats	350- 650	-	65-80	21-23	8.0
	800-1200	-	90	24	10-11
	1200-3500	-	95-105	25-30	12-14
	2000-4500	-	125-150	30-34	14-15
Open Hopper Barges	-	2000	175	26	9
	-	3000	195	35	9
	-	6000	290	50	9
Covered Dry Cargo Barges	-	2000	175	26	9
	-	3000	195	35	9
Liquid Cargo (Tank) Barges	-	2000	175	26	9
	-	1500	195	35	9
	-	3000	290	50	9
Deck Barges	-	700	110	26	6
	-	1800	130	30	7
	-	2400	195	35	8
Carfloats	-	-	257	40	10
	-	-	366	36	10
Scows	-	700	90	30	9
	-	2000	120	38	11
	-	2700	130	40	12

TABLE 2: REPRESENTATIVE CONTAINER SHIPS

(Seagoing Vessels)

Tonnage DWT (kips)	Displacement (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)	No. of Containers (circa)
112000	164640	951	106.3	42.7	2800
80530	114240	888	104.3	38.4	2000
56000	76160	696	98.4	35.1	1380
33600	44800	591	86.9	29.5	810
15680	21504	469	62.3	21.3	3.6

TABLE 3: SEAGOING FISHING VESSELS

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
5600	-	6272	295	45.9	19.4
4480	-	5600	279	42.7	18.4
3360	-	4704	262	39.4	17.4
2240	-	3920	246	36.1	16.4
1792	-	3472	230	34.4	15.7
1344	-	2688	213	32.8	14.8
896	-	1792	180	27.9	13.1
448	-	896	131	23.0	11.5

TABLE 3A: SEAGOING PASSENGER VESSELS

Tonnage (kips) GRT	DWT	Displacements (kips)	Overall Length (ft)	Length between Perps (ft)	Beam (ft)	Draft (ft)
179200	-	16800	1033	968	116.5	37.7
156800	-	145600	1033	968	111.5	36.1
134400	-	123200	1017	951	106.6	34.4
112000	-	100800	984	919	101.7	34.4
89600	-	78400	869	804	96.8	32.8
67200	-	67200	755	689	91.9	32.8

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TABLE 4: REPRESENTATIVE MIXED CARGO FREIGHTERS

(Full Deck Construction Seagoing Vessels)

Tonnage (kips) GRT	DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
22400	33600	44800	541	70.5	31.2
16800	24640	33600	492	65.6	29.5
11200	16800	22400	443	57.4	26.2
8960	13440	17920	394	52.5	24.6
6720	10080	13440	344	47.6	23.0
4480	6720	8960	312	42.7	19.7
3360	4928	6720	295	39.4	18.0
2240	3360	4480	246	34.4	14.8
1120	1568	2240	197	27.9	11.5

TABLE 5: REPRESENTATIVE BULK CARGO FREIGHTERS

(Ore, Oil, Coal, Grain, etc. Seagoing Vessels)

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
-	2240000	2564800	1677	228.7	106.6
-	1568000	1803200	1545	259.2	95.1
-	1008000	1173760	1391	224.7	82.0
-	761600	896000	1306	205.1	75.5
-	504000	604800	1168	175.5	67.3
-	280000	347200	968	142.7	52.5
-	100800	134400	755	95.1	37.7
-	56000	67200	623	80.4	34.4

GRT - Gross Registered Tonnage

DWT - Dead Weight Tonnage

for approaching vessels, as well as effect of wind, ice, current, or tide in the vicinity.

Depth of water may dictate the type of protection to be chosen. If the depth is so great, or the character of the waterway bottom does not lend itself to proper anchorage and support for an independent protective system, it may be necessary to design a suspended or floating protective system.

Types of Construction

The type of construction to be chosen for the protective system should be based on the physical site conditions and the amount of energy to be absorbed or deflected, as well as the size and ability of the pier itself to absorb or resist the impact.

Some of the more common types of construction are as follows:

Integral

Where the pier is considered to be stable enough to absorb the impact of floating vessels, it may be necessary to attach cushioning devices to the surfaces of the pier in the areas of expected impact to reduce localized damage such as spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing. Such cushioning may include strips of material attached to the face of the pier, such as solid rubber, timber, rubber-pneumatic, hydraulic or hydrocushion strips.

Dolphins

Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

Cellular dolphins may be filled with concrete, losse material or material suitable for grouting. Cells filled with uncemented materials may lose fill material in the event of rupture due to collision.

Floating Sheer Booms

Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

Hydraulic Devices

Suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier, or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

Fenders

Construction of fender systems, using piling with longitudinal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

Other Types

Various other types of protective systems have been successfully used and may be considered by the engineer.

Permits

Proposed protective systems must receive approval of the U.S. Coast Guard and probably other regulating agencies prior to installation. Advance handling with these agencies to determine waterway clearance, lighting and any other special requirements, is recommended.

DESIGN

General

Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

The location of the protective system (regardless of the type of construction) with respect to the navigation channel limits, stream current, prevailing winds, water depth, and normal water traffic approach angle is extremely important. The protective system should be located so that it will not hinder the vessel in negotiating the bridge opening, insofar as it is practical to do so.

In any type of pier protection system, general details should be designed to provide the following:

a. Replacement of damaged parts.

b. Elimination of sparking upon vessel impact.

c. Adequate mass and resilience so that the railroad facility will not be vulnerable to damage from normal collision of marine traffic.

Design Loads

Design loads to be used shall be determined for each individual structure, based on factors peculiar to the location. Information may be available from ship owners and operators, port facility authorities, industry representatives, the U.S. Army Corps of Engineers, and the U.S. Coast Guard.

General factors to be considered in determining the desired degree of pier protection include, but are not limited, to, the following:

a. Piers at the edge of a channel having wide horizontal clearance may require only minimum protection.
b. The type of construction of the pier should be considered.
    (1) A massive pier may be capable of resisting most anticipated loads so that the additional resistance offered by a protective system may not be warranted.
    (2) A pier incapable of resisting anticipated loads should be provided with greater protection than a massive pier might require.
c. Piers may be especially vulnerable because of difficulty of navigation caused by high stream velocity or tidal flow, wind velocity, limited horizontal clearances, channel curvature, proximity of other obstacles, or other similar factors.

d. Foundation conditions will have a bearing on the resistance capability of the pier and on the practicality of providing the desired degree of protection.

e. The history of collisions with existing bridges or other obstacles in the vicinity should be considered.

To determine the actual collison forces which could be encountered, and their effects, the following items should be known:

(a) Maximum sizes and types of vessels from Tables 1-5.

(b) Impact velocity of vessels.

(c) Crushing resistance of hulls.

(d) Stream velocities.

(e) High and low water elevations.

(f) Impact angle (Fig. A)

(g) Wind velocities.

(h) Velocity and mass of floating ice.

Horizontal Live Load

A. Force-Acceleration Method

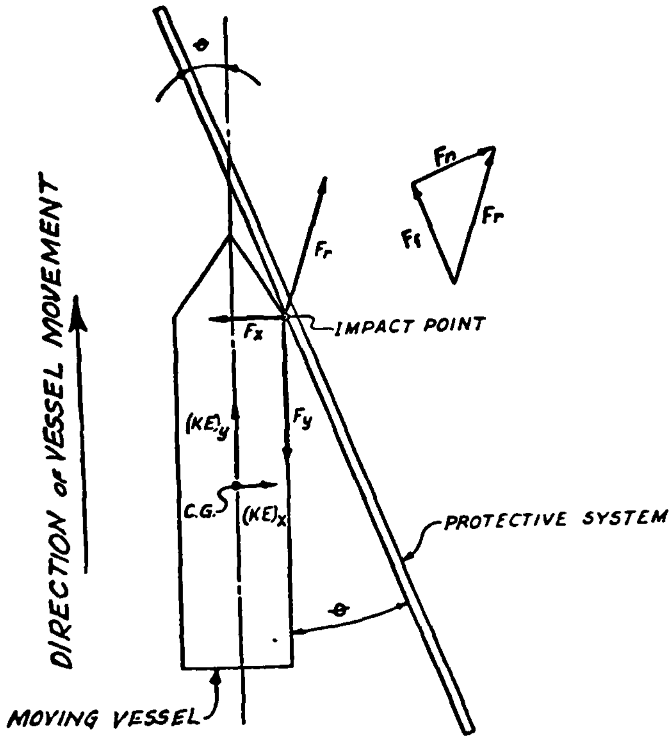
The applied horizontal force to an individual pile, fender or dolphin is computed from:

$$P = K_e YC \tag{1}$$

where:

P = Applied Horizontal force

$$K_e = \frac{K_p \cdot K_f}{K_p + K_f}$$



F_x = FORCE ON VESSEL LATERALLY
F_y = FORCE ON VESSEL AXIALLY
$(KE)_x$ = INERTIA FORCE (KINETIC ENERGY) LATERALLY
$(KE)_y$ = INERTIA FORCE (KINETIC ENERGY) AXIALLY
F_r = RESULTANT FORCE ON PROTECTIVE SYSTEM
F_f = FRICTIONAL OR TANGENTIAL FORCE ON PROTECTIVE SYSTEM
F_n = NORMAL FORCE ON PROTECTIVE SYSTEM
θ = IMPACT ANGLE
C_g = CENTER OF GRAVITY

FIGURE A

$K_p = (L_p^3/3\ EI_p)\,D.F.$

K_f = Fender, stiffness ($01. \leq K_f \leq 60$, average of 30)
If $K_f = 0$ then $K_e = K_p$

L_p = Length of pile

E = Modulus of elasticity of the material

I_p = Moment of inertia of the pile

D.F. = Distribution Factor

$Y = V_o/\lambda$

V_o = Initial velocity of vessel(in/sec)

$\lambda = (K/M)^{1/2}$

$M = W_S/32.2\,(\mathrm{ksec}^2/\mathrm{in})$

W_S = Weight of the vessel

$C = C_E \cdot C_C \cdot C_H$

C_E = Eccentricity Coefficient
where C_E is determined from Fig.1

C_C = Configuration Coefficient
where C_C equals

Pier Type	C_C
Open	1.0
Semi-Closed	0.9
Closed	0.8

C_H = Hydrodynamic Mass Coefficient = $1 + 2\,D/B$

D = Draft of vessel

B = Beam of vessel

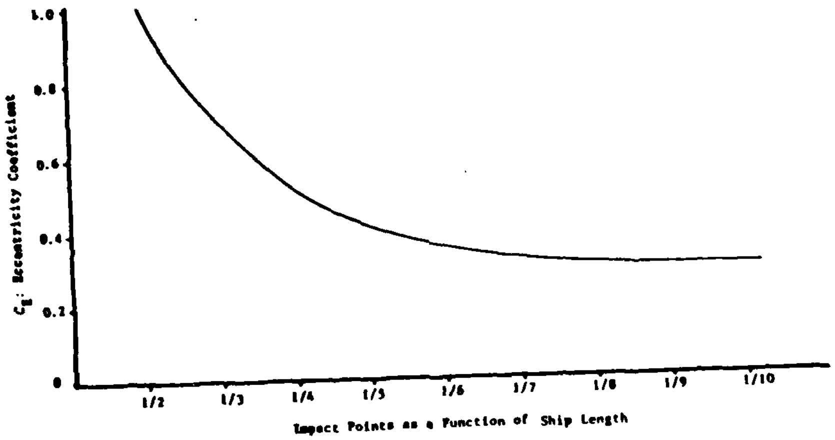


Fig 1: Eccentricity Coefficient vs. Impact Points

The resulting acceleration and stopping time is computed from

$$a = V_o \lambda \qquad (2)$$

$$t = \pi/2 \lambda \qquad (3)$$

Thus, with the use of the above equations any fender, dolphin, or protective system can be designed. The difference from normal design criteria (which has proved to be inadequate) is the use of the distribution factor. The following equation may be used to determine the D.F.

$$D.F. = [-6.0x10^{-7} D_x+F] L_p^{-0.006} \qquad (4)$$

where:

$$F = -3.5x10^{-13}(D_y)^2+3.1x10^{-7} D_y+0.335$$

$$D_y = \frac{El_p}{S_p} \text{ (vertical pile stiffeners)}$$

$$D_x = \frac{El_\omega}{S_\omega} \text{(transverse stiffeners of walers)}$$

S_p = Spacing between piles (in.)

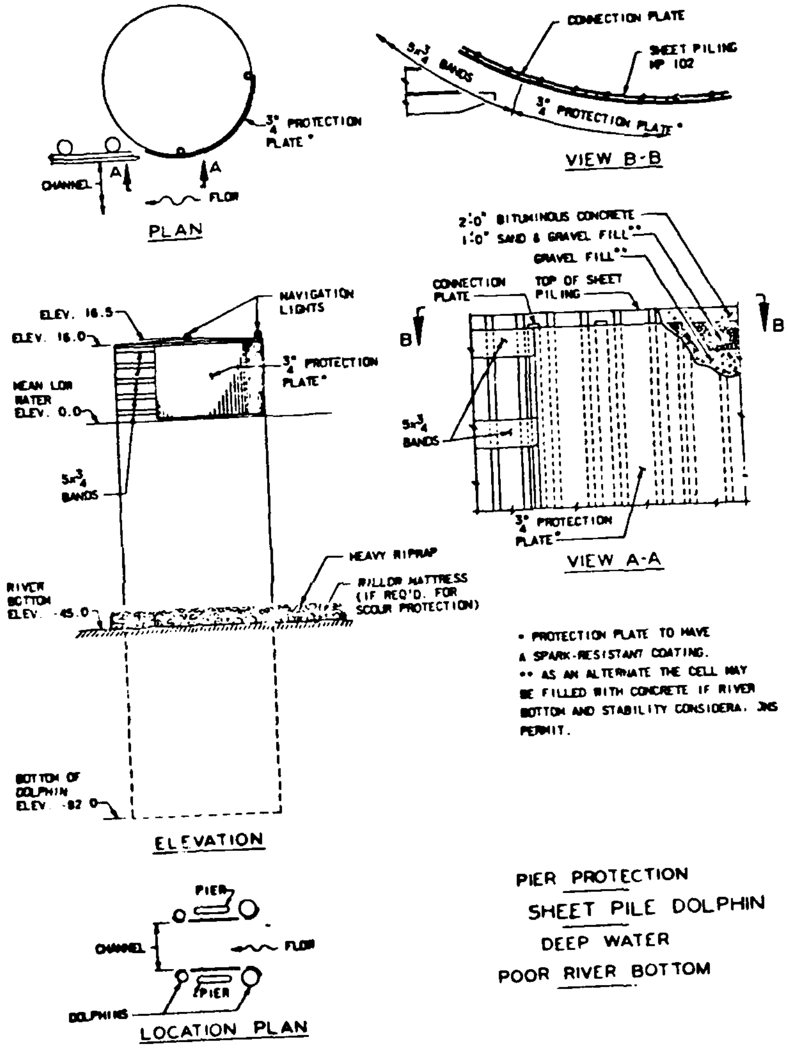
S_ω = Spacing between walers (in.)

Types of Protection

The following types of protection are commonly used; however, other types may be considered.

Sheet Pile Cell Dolphins (see fig.B)

Sheet pile cells preferably should be of circular configuration. A typical cell includes interlocking steel sheet piles filled with concrete or grouted material. If loose fill an opening to allow for adding additional fill should be provided. The concrete top should



CONNECTION PLATE
SHEET PILING MP 102
5x3/4 BANDS
3/4" PROTECTION PLATE *
VIEW B-B
3/4" PROTECTION PLATE *
CHANNEL
A
A
FLOW
PLAN
2'-0" BITUMINOUS CONCRETE
1'-0" SAND & GRAVEL FILL**
GRAVEL FILL**
CONNECTION PLATE
TOP OF SHEET PILING
B
B
5x3/4 BANDS
3/4" PROTECTION PLATE*
VIEW A-A
ELEV. 16.5
ELEV. 16.0
NAVIGATION LIGHTS
3/4" PROTECTION PLATE*
MEAN LOW WATER ELEV. 0.0
5x3/4 BANDS
HEAVY RIPRAP
WILLOW MATTRESS (IF REQ'D. FOR SCOUR PROTECTION)
RIVER BOTTOM ELEV. -45.0
BOTTOM OF DOLPHIN ELEV. -82 0
ELEVATION
* PROTECTION PLATE TO HAVE A SPARK-RESISTANT COATING.
** AS AN ALTERNATE THE CELL MAY BE FILLED WITH CONCRETE IF RIVER BOTTOM AND STABILITY CONSIDERA. JNS PERMIT.
PIER
CHANNEL
FLOW
PIER
DOLPHINS
LOCATION PLAN
PIER PROTECTION
SHEET PILE DOLPHIN
DEEP WATER
POOR RIVER BOTTOM

FIGURE B

be adequately anchored to the sheet piles. Desirable qualities of ill material include free draining characteristics, high unit weight, shear strength, and high coefficient of friction.

The designer should make an evaluation of the cell stability and resistance to overturning and sliding. Factors to be considered include characteristics of the underlying soil or rock and the cell fill material, interaction of the cell fill material with the cell walls, and friction of the sheet piles embedded in the underlying soil.

Additional resistance against overturning may be provided by driving and attaching additional piles around the perimeter of the cell. Increased penetration into the underlying soil may be obtained in this manner, in lieu of extension of all sheet piles.

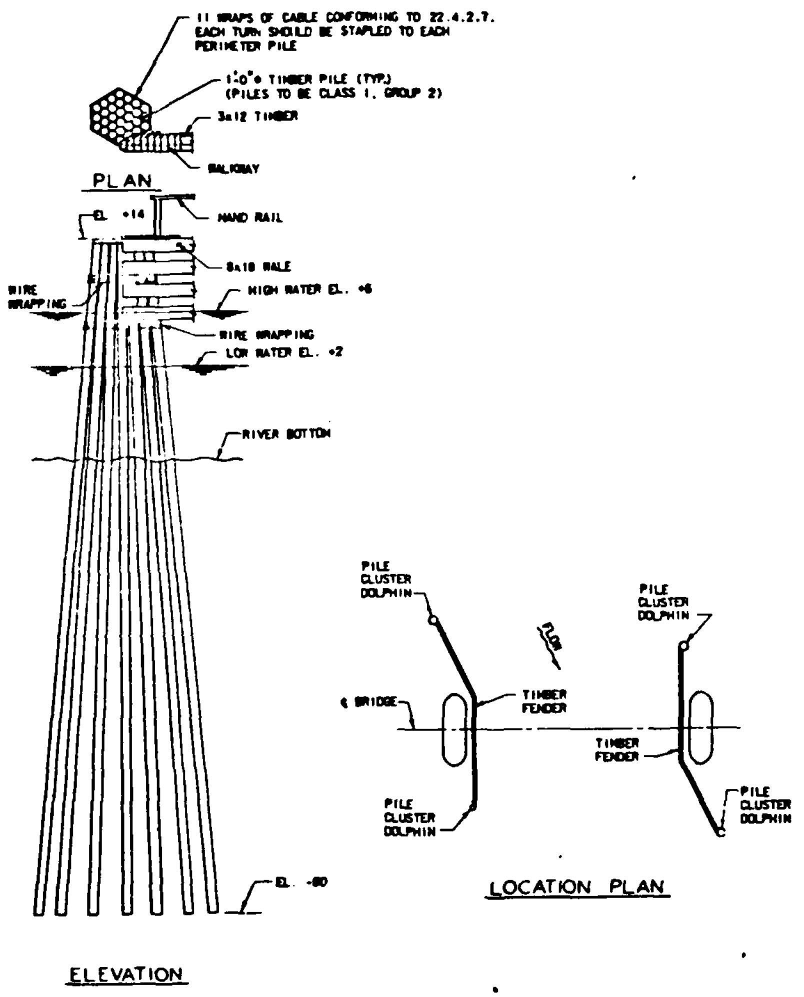
The possibility of scour occurring near a dolphin should be investigated and protection should be provided, if required.

Pile Cluster Dolphins (see fig.C)

Pile cluster type dolphins should be composed of groups of battered and/or vertical piles which are held together at the top. The designer should evaluate the resistance to lateral forces, considering the effects of any battered piles, and the interaction of the piles and the surrounding soils.

Gravity Pendulum Dolphin (Hydrocushion Type)(see Fig.D)

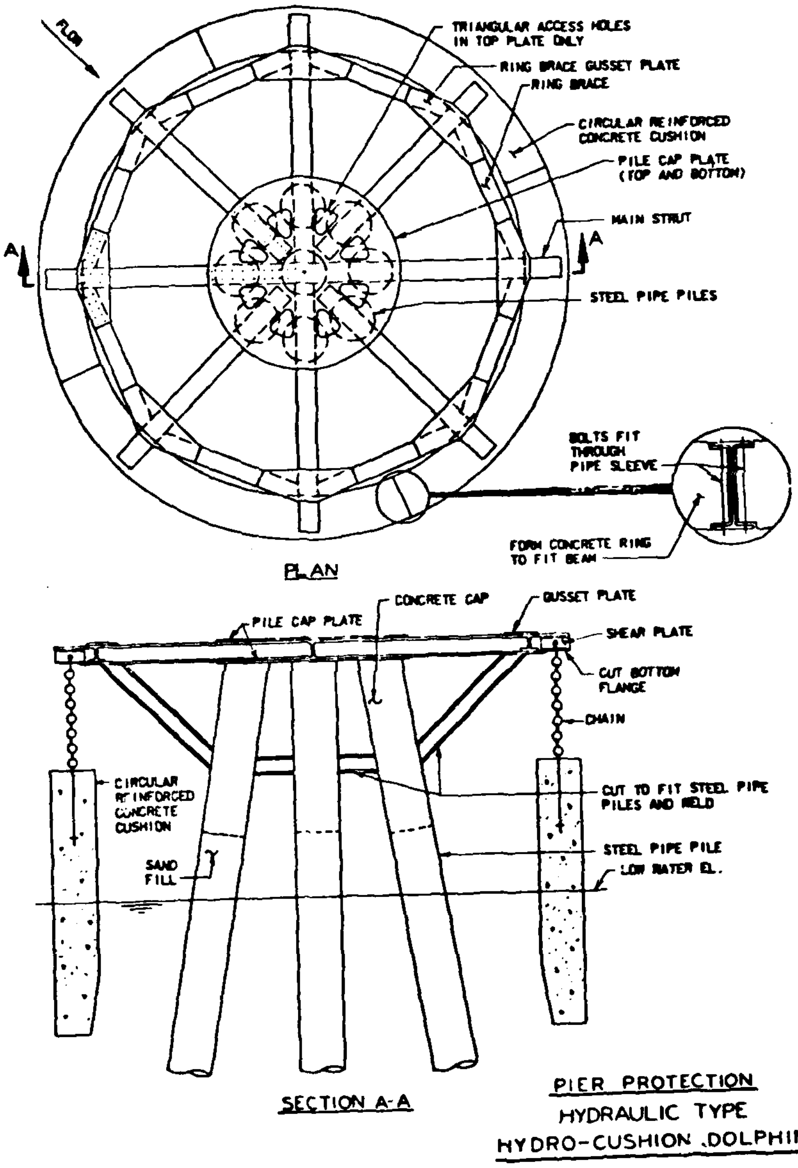
Typically, a heavy cylindrical mass of steel or concrete is suspended from a cantilevered supporting structure, which may be a part of the pier, or may be an independent support. Energy is dissipated by movement of the pendulum when a force is applied by a striking vessel.



PIER PROTECTION

TREATED TIMBER PILE DOLPHIN

FIGURE C



FLOW
TRIANGULAR ACCESS HOLES IN TOP PLATE ONLY
RING BRACE GUSSET PLATE
RING BRACE
CIRCULAR REINFORCED CONCRETE CUSHION
PILE CAP PLATE (TOP AND BOTTOM)
MAIN STRUT
STEEL PIPE PILES
A
A
BOLTS FIT THROUGH PIPE SLEEVE
FORM CONCRETE RING TO FIT BEAM
PLAN
CONCRETE CAP
GUSSET PLATE
PILE CAP PLATE
SHEAR PLATE
CUT BOTTOM FLANGE
CHAIN
CIRCULAR REINFORCED CONCRETE CUSHION
CUT TO FIT STEEL PIPE PILES AND WELD
STEEL PIPE PILE
LOW WATER EL.
SAND FILL
SECTION A-A
PIER PROTECTION
HYDRAULIC TYPE
HYDRO-CUSHION DOLPHIN

FIGURE D

The designer should evaluate the energy dissipated by the pendulum, taking the following items into account.

a. Movement of the pendulum. When the pendulum is suspended in water, the effective mass includes an amount of water which moves along with the pendulum: in the case of a ring, (as shown in Figure D) the volume of water enclosed by the ring is part of the total mass to be moved.

b. The resisting horizontal force component

b. The resisting horizontal force component = $Wr(\frac{x}{l-y})$

in which: Wr = Weight of the ring

x = The horizontal displacement of the ring

l = Length of hanger to the ring

y = The amount the ring is lifted

Floating Sheer Booms (see fig. E).

The configuration of a sheer boom will depend upon the requirements of a particular location.

The designer should evaluate the capability of the device to dissipate energy, recognizing the following:

a. The mass to be considered as part of the moving element includes a volume of water which will be forced to move with the boom.

b. Deflection movements of supporting elements will account for some energy loss.

c. Frictional resistance is provided by the water adjacent to the moving elements.

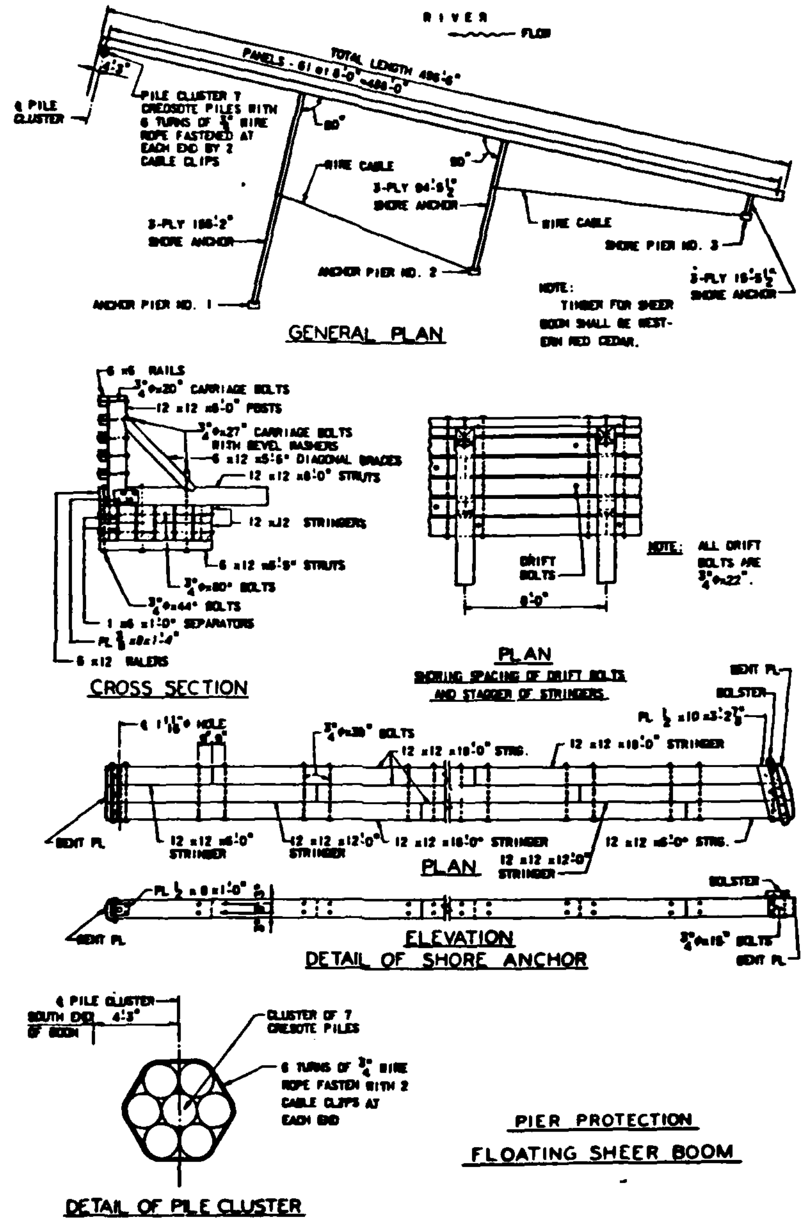

RIVER
FLOW
GENERAL PLAN
NOTE:
TIMBER FOR SHEER BOOM SHALL BE WESTERN RED CEDAR.
ANCHOR PIER NO. 1
ANCHOR PIER NO. 2
SHORE PIER NO. 3
WIRE CABLE
CROSS SECTION
PLAN
SHOWING SPACING OF DRIFT BOLTS AND STAGGER OF STRINGERS.
NOTE: ALL DRIFT BOLTS ARE
PLAN
ELEVATION
DETAIL OF SHORE ANCHOR
CLUSTER OF 7 CRESOTE PILES
DETAIL OF PILE CLUSTER
PIER PROTECTION
FLOATING SHEER BOOM

FIGURE E

Fenders (see figs. F,G,H and I).

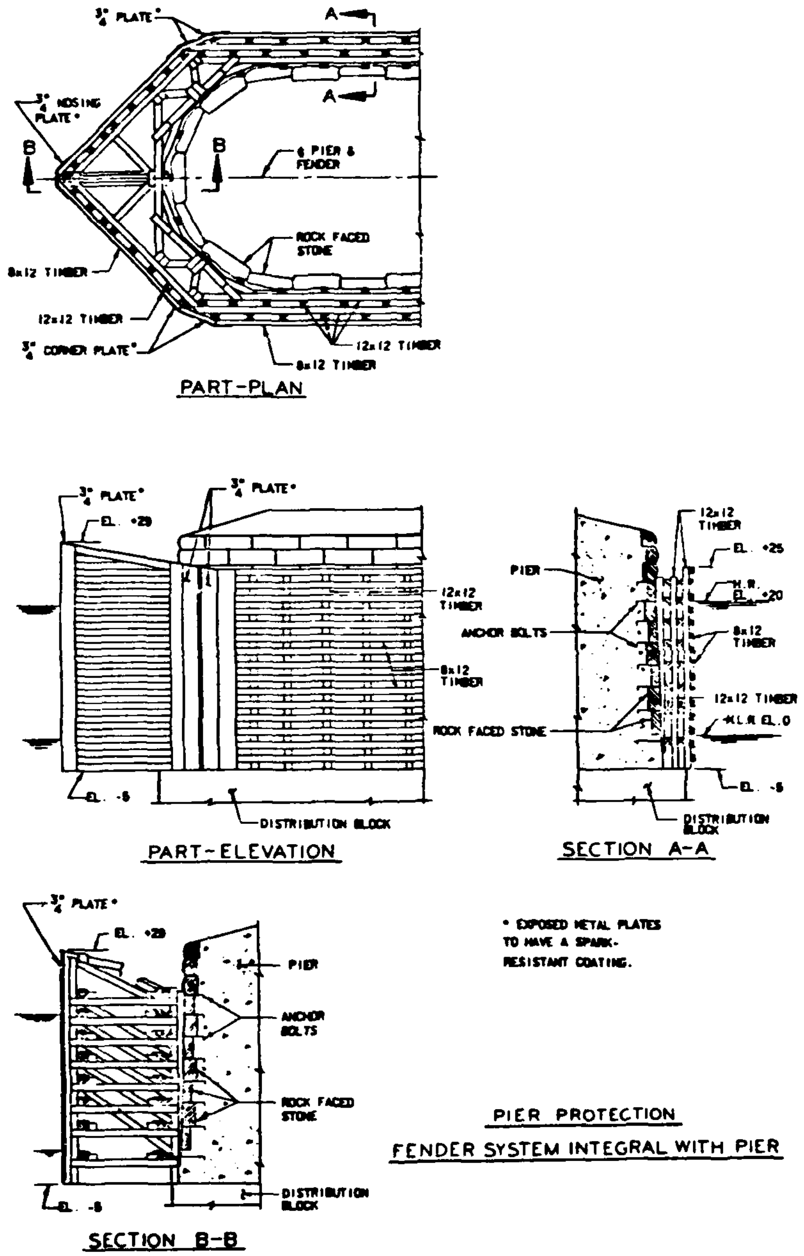
Pier fenders are constructed to provide for some degree of protection to the pier in the event of contact by a vessel. Fenders are usually positioned to anticipate the direction of impact from a vessel to be at a relatively small angle with respect to the fender line. A fender may be supported by the pier it is intended to protect, or it may be independently supported.

Independently supported fender systems typically consist of vertical and/or battered piles with horizontal members connecting the piles so the fender system act as a unit. The horizontal members may be used as rubbing strips or separate rubbing strips may be attached to these members.

Pier-supported fenders vary in type from simple rubbing strips attached directly to the pier face to more elaborate installations which provide for some energy dissipation by the fender when struck by a vessel.

The designer should consider the following items pertaining to fenders:

a. Fenders should preferably be detailed so that a maximum number of piles, or other supporting elements, will participate in resisting applied loads.
b. Generally, a somewhat flexible arrangement that provides for deflection movement of the fender is preferred to provide for energy dissipation.
c. The effects of battered piles and pile-soil interaction should be considered when evaluating the capability of the fender to resist lateral forces.



3/4" PLATE*
A
3/4" NOSING PLATE*
B
℄ PIER & FENDER
ROCK FACED STONE
8x12 TIMBER
12x12 TIMBER
3/4" CORNER PLATE*
12x12 TIMBER
8x12 TIMBER
PART-PLAN
3/4" PLATE*
EL. +29
3/4" PLATE*
12x12 TIMBER
8x12 TIMBER
EL. -5
DISTRIBUTION BLOCK
PART-ELEVATION
PIER
12x12 TIMBER
EL. +25
ANCHOR BOLTS
H.R. EL. +20
8x12 TIMBER
12x12 TIMBER
ROCK FACED STONE
M.L.R. EL. 0
EL. -5
DISTRIBUTION BLOCK
SECTION A-A
3/4" PLATE*
EL. +29
PIER
ANCHOR BOLTS
ROCK FACED STONE
EL. -5
DISTRIBUTION BLOCK
SECTION B-B
* EXPOSED METAL PLATES TO HAVE A SPARK-RESISTANT COATING.
PIER PROTECTION
FENDER SYSTEM INTEGRAL WITH PIER

FIGURE F

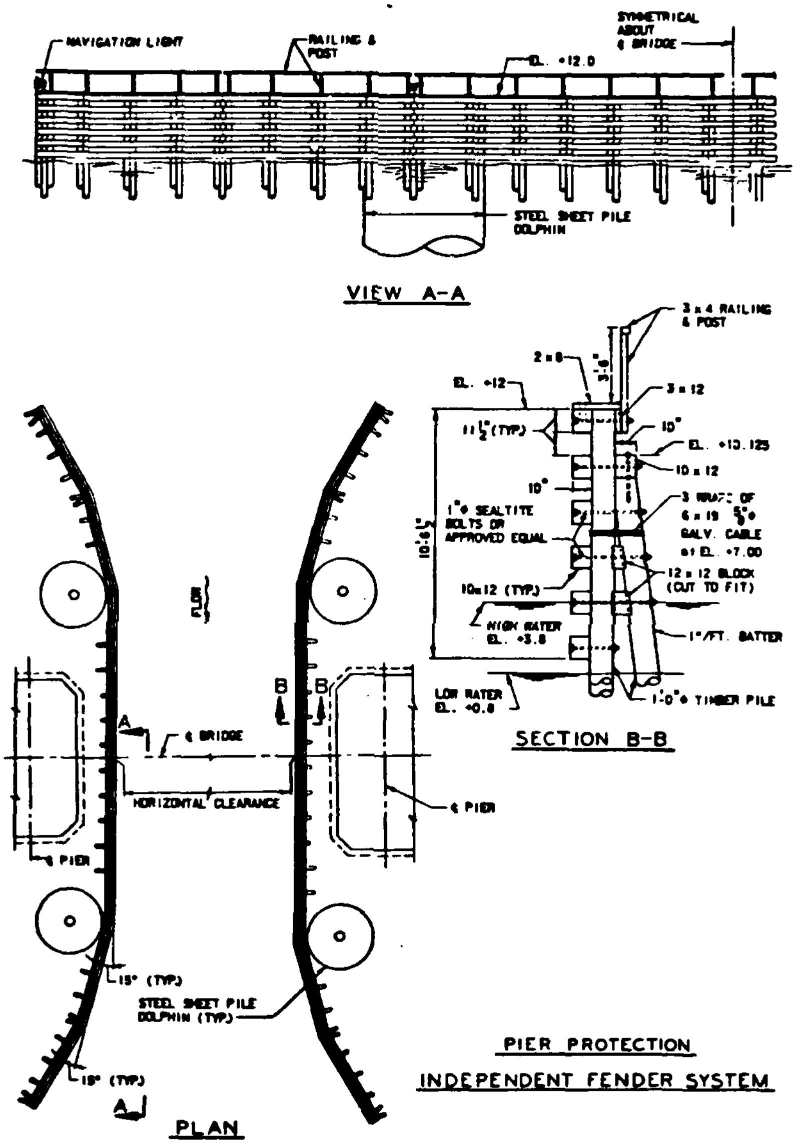
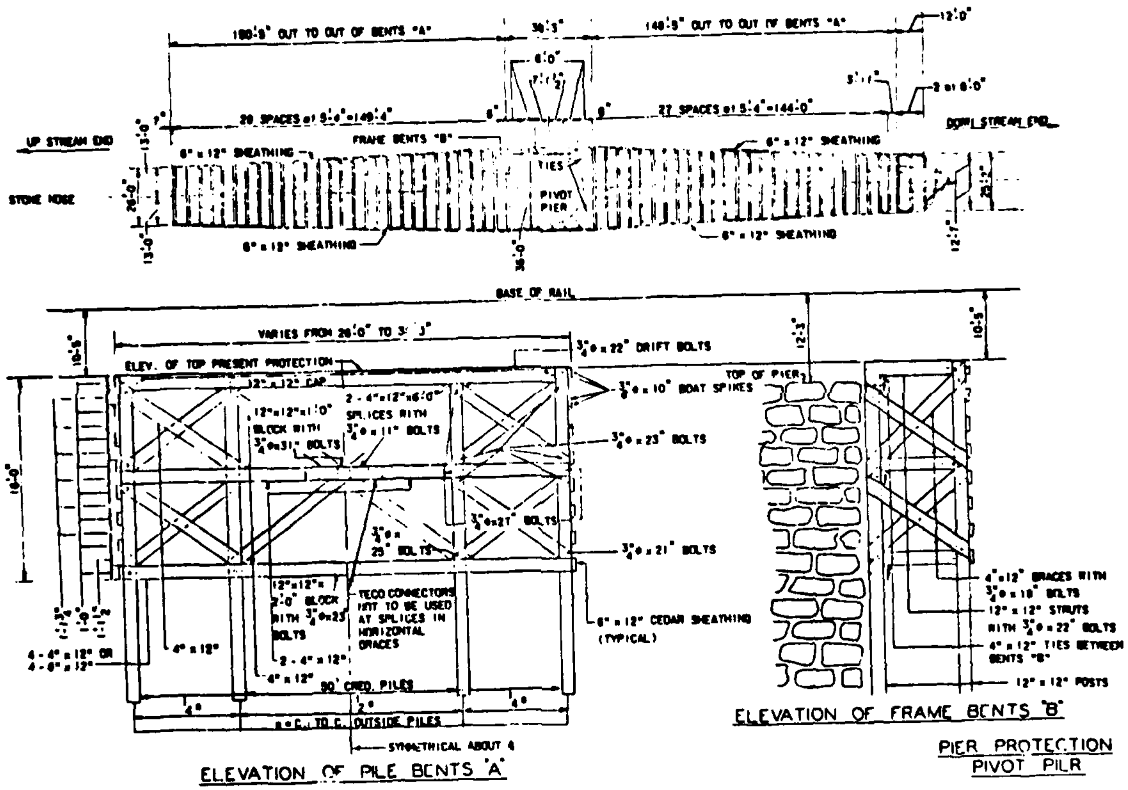
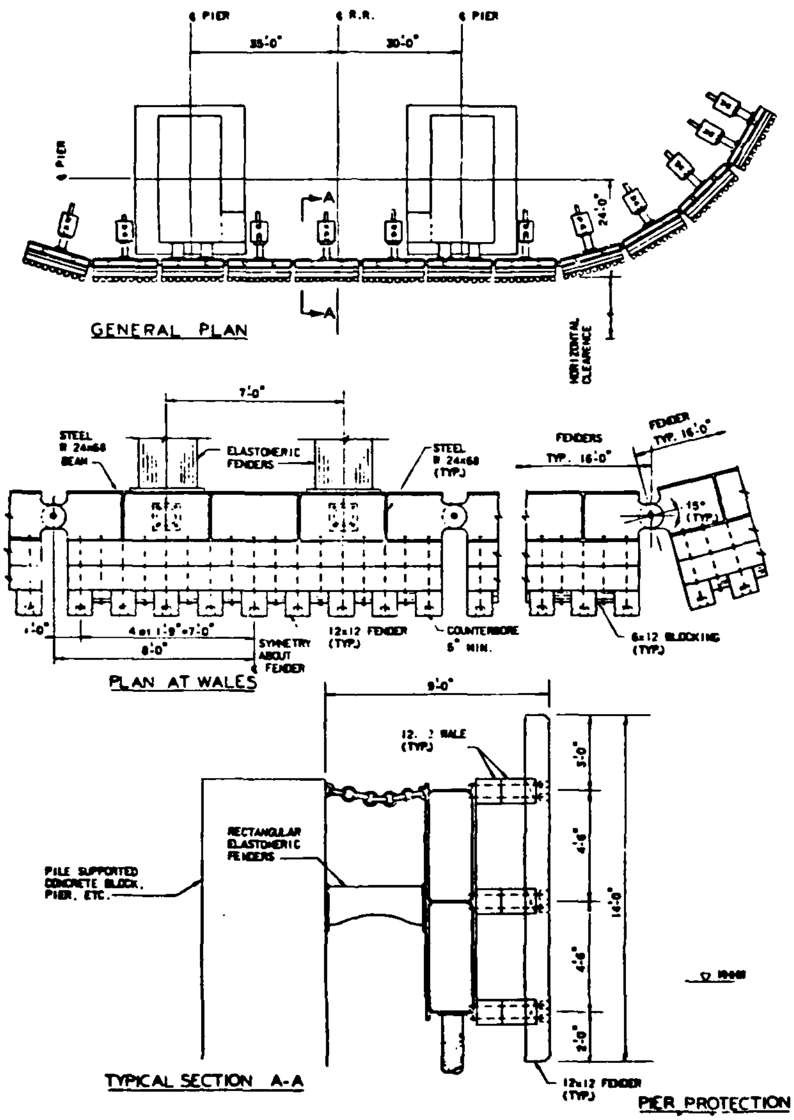

NAVIGATION LIGHT
RAILING & POST
EL. +12.0
SYMMETRICAL ABOUT ℄ BRIDGE
STEEL SHEET PILE DOLPHIN
VIEW A-A
3 x 4 RAILING & POST
2 x 8
EL. +12
3 x 12
10"
EL. +13.125
10 x 12
1"φ SEALTITE BOLTS OR APPROVED EQUAL
GALV. CABLE
12 x 12 BLOCK (CUT TO FIT)
10x12 (TYP)
HIGH WATER EL. +3.8
1"/FT. BATTER
LOW WATER EL. +0.8
1'-0"φ TIMBER PILE
SECTION B-B
FLOW
A
B
℄ BRIDGE
HORIZONTAL CLEARANCE
℄ PIER
15° (TYP)
STEEL SHEET PILE DOLPHIN (TYP)
PLAN
PIER PROTECTION
INDEPENDENT FENDER SYSTEM

FIGURE G



UP STREAM END
STONE NOSE
DOWN STREAM END
FRAME BENTS "B"
TIES
PIVOT PIER
6" x 12" SHEATHING
BASE OF RAIL
ELEV. OF TOP PRESENT PROTECTION
12" x 12" CAP
TOP OF PIER
4" x 12"
90' CREO. PILES
TECO CONNECTORS NOT TO BE USED AT SPLICES IN HORIZONTAL BRACES
6" x 12" CEDAR SHEATHING (TYPICAL)
12" x 12" STRUTS
12" x 12" POSTS
SYMMETRICAL ABOUT ℄
ELEVATION OF PILE BENTS 'A'
ELEVATION OF FRAME BENTS 'B'
PIER PROTECTION
PIVOT PIER

FIGURE H



¢ PIER
¢ R.R.
¢ PIER
35'-0"
30'-0"
24'-0"
GENERAL PLAN
HORIZONTAL CLEARENCE
7'-0"
STEEL
ELASTOMERIC FENDERS
FENDERS TYP. 16'-0"
12x12 FENDER (TYP.)
COUNTERBORE 5" MIN.
6x12 BLOCKING (TYP.)
4 at 1'-9"=7'-0"
8'-0"
SYMMETRY ABOUT ¢ FENDER
PLAN AT WALES
9'-0"
RECTANGULAR ELASTOMERIC FENDERS
PILE SUPPORTED CONCRETE BLOCK, PIER, ETC.
14'-0"
TYPICAL SECTION A-A
12x12 FENDER (TYP.)
PIER PROTECTION

FIGURE I

d. Consideration should be given to providing a weak point in the design, thus causing the unit to fail in a pre-planned manner when struck by a force in excess of the capacity. Details can then be arranged to facilitate the replacement of damaged elements.

Riprap Used as Pier Protection

Pier which are located near the shoreline or in shallow water at the edge of a ship channel may require minimum protection. Riprap may be deposited near a pier for the purpose of preventing erosion and to reduce the water depth, thus protecting the pier from vessel by stopping them before contact is made.

COMMENTARY ON PIER PROTECTION SYSTEMS AT SPANS OVER NAVIGABLE STREAMS

Energy Dissipation

A moving vessel has a certain amount of kinetic energy, which is dependent upon the mass of the vessel and its velocity. If we are to redirect or stop this vessel in protecting the pier, a portion or all of this kinetic energy must be absorbed or dissipated. This energy is dissipated by applying a force to the vessel over a given distance. For the fender to function properly, this distance must be less than the distance from initial contact until the vessel would strike the pier. For large vessels, traveling at fair speeds, in deep water, the amount of kinetic energy provided is large and the resistance of the fender is relatively small and it is very difficult to design a fender that will completely protect a pier for such a collision if the vessel is headed directly at the pier.

The energy in any contact with the fender is disspated by deflection of the fender itself, by lifting a portion of the fender, by lifting the vessel out of the water, by crushing of the fender, by crushing of the bow of the vessel, by displacement of the water adjacent to the vessel, by displacement of the ground or river bottom, etc.

Several general facts should be considered and are noted briefly:

1. It should be recognized that the total resisting force is not developed immediately upon impact, but requires some movement until it develops.
2. If the crushing force of the vessel is greater than the ultimate resisting force of the fender, then dissipation of the kinetic energy occurs in two phases. In the first phase, the impact creates a force between the vessel and the fender, which causes the vessel to decelerate and the fender to accelerate (F = mass x acceleration). At some point, the fender and the vessel reach the same velocity and move along together, being slowed by the resisting forces of the fender and/or the soil being acted upon. This will continue until either the vessel stops, the fender breaks or some combination of the two.
3. If the crushing force of the vessel is less than the total ultimate resisting force of the fender, then the velocity of the fender will increase from zero to a maximum and decrease to zero again without a common velocity being achieved. When the fender stops, the vessel continues to decelerate, acted upon by the crushing force.

Fender Flexibility

An ideal pier fender would be constructed so that the fender itself absorbs all of the energy of the moving vessel in stopping the vessel before it hits the pier and then returns to its normal position without damage to either the fender or the vessel. Except for relatively small vessels and low speeds, design of such a fender is impractical due to the large required resisting force and the short distance in which to stop the vessel.

A flexible fender, one that acts elastically, will absorb energy with little or no damage to the vessel; however, the horizontal force that such a fender can resist is usually relatively small and may be insufficient to protect the pier. On the other hand, a rigid fender is capable of resisting a considerably larger force, although this force may only be applied over a small deflection before the member breaks, or is damaged locally. In this case, the total amount of energy absorbed may be far less than is absorbed in a flexible fender, although a considerasle amount of energy is absorbed in breaking of the fender parts. In most cases, some compromise between a truly flexible and a very rigid fender is the better solution.

In fender systems, incorporating steel pipe piles or sheet pile cells, a concrete fill will provide a much more rigid device than will one filled with sand or stone or riprap. In the latter case, the energy absorbing qualities are improved due to the rubbing of the fill particles on each other, by friction in the interlocks of the sheet piles and the like. On the other hand, one must be extremely careful that the pile wall or the sheet pile wall is protected to prevent damage resulting in the loss of fill, which materially reduce the effectiveness of the fender and its energy absorbing capability. The sand filled pipe is much more likely to deflect and to bend than the concrete filled pipe, which will only deflect a small distance before shearing-off.

The type of fender used in any particular application must take into account the size and velocity of the vessel, flow of the stream, the depth of the water, the founding conditions, the distance between the pier protection and the pier, the strength of the pier itself and the types of cargo that are normally carried. The engineer must normally use his discretion in selecting a pier protection design that best suits all of the parameters of the individual case considered.

Sources of Information

Stream velocities for various river stages on most navigable waters can be obtained from the U.S. Corps of Engineers, Channel locations, navigation maps and scour potential, may be available from the U.S. Corps of Engineers and the U.S. Coast Guard.

Information regarding principal sizes, capacities and power of various vessels, as well as the type of cargo is usually available for navigable waters from the U.S. Corps of Engineers, the U.S. Coast Guard, the American Waterways Operators, Inc., ports authorities, pilots associations and others.

Specific site parameters such as, riverbed conditions, soil information, local wind and current effects on navigation usually must be developed by the design engineer, although local pilots associations and waterway users associations may be able to help with the latter.

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CHAPTER V: AASHTO STANDARDS

INTRODUCTION

The changing character and volume of marine transportation presents new and increasing demands on our waterways and adjacent structures such as bridges, wharves, harbor piers, marinas, and lock entrances. These demands are attributed to the phenomenal growth in navigation module (size and speed) whether it be a tanker, containership or barge tows. Because of these increases in navigation module, the forces that can be delivered to structures adjacent to the waterway have substantially increased.

Conceptually, protection of the vessels, bridge piers, and adjacent structures such as wharves is provided by a fender system adjacent to the navigation opening of the bridge or along the wharf, rather than the vessel. The dilemma which now exists is simply that vessels have grown in magnitude and disproportionately to the growth in size or capability of the protective systems.

Tankers built during the early part of this century had an overall length of about 90 to 150 m (300 to 500 ft) and displacements as light as 5,000 long tons. Currently tankers are being built greater than 300 m (1,000 ft) with displacements of more than 400,000 long tons. Bulk carriers have grown from 120 m (400 ft) to over 200 m (650 ft) with displacements of 20,000 to 22,000 long tons. Additionally, barges moving on the Gulf Intracoastal Waterway now measure up to 90 m (300 ft) and have a liquid capacity of 5,000,000 liters, 31,000 bbl or 3,000 long tons.

Coast Guard casualty statistics show that vessel collisions with fixed objects more than doubled between 1966 and 1975 as larger and greater numbers of vessels used the nation's waterways. One Coast Guard study reveals that from 1970 through 1974 at least 811 accidents occurred at bridges, which resulted in over $23 million in damage and 14 fatalities. Obviously, such statistics indicate that a need exists to assure that proper design practices and construction techniques are used for protective systems and devices, as well as proper design criteria and standards.

Factors Considered in the Design

The function of bridge fendering systems is to protect bridge elements against damage from waterborne traffic. There are many factors to be considered in the design of fendering systems including the size, contours, speed and direction of approach of the vessels using the facility; the wind and tidal current conditions expected during the ship's maneuvers and while tied up to the berth; and the rigidity and energy absorbing characteristics of the fendering system and ship.

The final design selected for the fender system will generally evolve after reviewing the relative costs of initial construction of the fendering system versus the cost of fender maintenance and of ship repair. In other words, it will become necessary to decide upon the most severe docking or approach conditions to protect against and design accordingly; hence, any situation which imposes conditions more critical than the established maximum would be considered in the realm of accidents and probably result in damage to the dock, fendering system (whether used for dock or approach conditions) or the ship.

BRIDGE AND WHARF PROTECTIVE SYSTEMS

Many fendering systems have been designed and/or analyzed.[1-10] These systems are of wide variety and material which vary considerably in design, fabrication and cost. As a result of a literature survey it appears that basically seven types of fendering systems are in existence. These seven systems are as follows:

1. Floating Fender or Camel
2. Standard Pile-Fender System
    a. Timber Pile
    b. Hung Timber
    c. Steel Pile
    d. Concrete Pile
3. Retractable Fender System
4. Rubber Fender System
    a. Rubber in Compression
    b. Rubber in Shear
    c. Lord Flexible
    d. Rubber in Tension
    e. Pneumatic
5. Gravity Type Fender System
6. Hydraulic and Hydraulic-Pneumatic Fender System
    a. "Dashpot" Hydraulic
    b. Hydraulic-Pneumatic Floating Fender
7. Spring Type Fender System

In many situations the fendering systems listed are used along piers or wharves. They appear in bridge pier protection when used in combination with and attached to a series of driven piles, which get lateral support from walers. This use is becoming more frequent as the size of vessels increases, leaving less channel width for dolphins, cells and platform systems for the protection of structures.

The floating fender or camel is the simplest type of fendering system employed. In addition to a floating unit, it requires horizontal and/or vertical timber members bolted to the face of the wharf structure. The vertical may or may not be driven as piles. This type of fendering system was applicable prior to the 1930s. However, with the advent of the larger merchant vessels, particularly the bulk carriers and with the construction of docking facilities in relatively exposed locations, this system has been outdated.

The timber-pile system employs piles driven into the bottom along a wharf face. Pile tops may be unsupported laterally or supported at various degrees of fixity by means of walers and chocks. Single or multiple row walers may be used, depending on pile length and on tidal variation. Impact energy, upon a timber fender pile, is absorbed by deflection and the limited compression of the pile. Timber piles are abundant and have a low initial cost. They are susceptible to mecha ical damage and biological deterioration. Once this happens, the energy absorption capacity declines and a high maintenance or repair cost results.

The hung timber system consists of timber members fastened rigidly to the face of a dock. A contact frame is formed which distributes impact loads, but its energy-absorption capacity is limited and it is unsuitable for locations with significant tide and current effects. The hung-timber system has a low initial cost and is less bio-deterioration hazard than the standard timber pile.

Steel fender piles are occasionally used in water depths greater than 40 ft, or for locations where very high strength is required and a difficult seafloor condition results. Its main disadvantages are high cost and its vulnerability to corrosion.

Precast regular reinforced concrete piles are not satisfactory because of their limited internal strain-energy capacity; the reinforcement may corrode as moisture reaches the steel due to concrete cracking. Prestressed concrete piles with rubber buffers at deck level have been used. In this case, the rubber units are the principal energy-absorbing media, and not the piles. This system is very resistant to natural and biological deterioration.

Retractable fendering systems consist of vertical-contact posts connected by rows of walers and chocks. Contact posts are normally spaced 8 ft on center. The spacing between walers is dependent on local tide range. Walers are fastened to holding posts suspended by pins from specially designed brackets. The fender retracts under impact, thus ab·orbing energy by action of gravity and friction.

Energy absorption capacity depends directly on the effective weights, the angle of inclination of the supporting brackets, and the maximum amount of retraction of the system. In designing this system, tide effect on weight reduction of the fender frame should be considered.

Use of composite inclined planes of supporting brackets and proper selection of maximum retraction are feasible means of attaining design capacity. Fenders are more easily removed from open pin brackets than from slot type. In construction, the supporting brackets should be adequately anchored to the associated berthing structure. Although retractable fenders have a high initial cost, they have a low maintenance cost with minimum time loss during replacement.

Rubber in Several Configurations

Rubber in compression systems consists of a series of rubber cylindrical or rectangular tubes installed behind standard fender piles or behind hung-type fenders. Energy absorption is achieved by compression of the rubber. Absorption capacity depends on the size of the buffers and on maximum deflection. In design, a proper bearing timber-frame is required for transmission of impact forces from ship to pier.

Draped rubber tubes hanging from solid wharf bulkheads may be used; however, this solid wall should be at least a 3 ft vertical contact with the ship's hull. Energy absorption capacity of such a system can be varied by using the tubes in single or double layers, or by varying tube sizes. The energy absorption of a cylindrical tube is nearly directly proportional to the ship's force until the deflection equals approcimately one-half the external diameter, after that, the force increases much more rapidly than the absorption of energy.

Rubber-in-shear (Raykin) consists of a series of rubber pads bonded between steel plates to form a series of "sandwiches" mounted firmly as buffers between a pile fender system and pier. Two types of mounting units are available, which are capable of absorbing 100 percent of the energy. The problem with rubber-in-shear fenders is that they tend to be too stiff for small vessels and the steel plates have a tendency to corrode. Therefore, it follows that they have high energy absorbing capacity for larger ships.

The Lord Flexible fender consists of an arch-shaped rubber block bonded between two end steel plates. It can be installed on open or bulkhead type piers and on dolphins, or incorporated with standard piles or the hung system. Impact energy is absorbed by bending and compression of the arch-shaped rubber column. With the Lord Flexible fender, possible destruction of the bond between the steel plates and rubber may result.

Rubber-in-torsion fenders consist of a combination of rubber and steel fabricated in a cone-shaped, compact bumper form, molded into a specially cast steel frame and bonded to steel. It absorbs energy by torsion, compression, shear and tension. Their main disadvantage is possible destruction of the bond between steel castings and the rubber.

The final category of rubber fendering systems is the pneumatic fenders. These are pressurized, air-tight rubber devices designed to absorb energy by compression of air inside a rubber envelop. Pneumatic fenders are not applicable to fixed dock-fender systems, but are feasible to use as ship fenders or shock absorbers on floating fender systems.

A proven fender of this type is the pneumatic tire-wheel fender which consists of pneumatic tires and wheels capable of rotating freely around a fixed or floating axis. The fixed unit may consist of two to five tires. Energy absorption capacity and resistance load depends on the size and number of tires used and on initial air pressure when inflated.

A recent development is the use of foam-filled fenders. In hard collisions there is a slight chance that the ordinary pneumatic fenders may be overloaded and release air through a safety valve or through a puncture. In this event the pneumatic fender becomes unoperable. Repair may be possible on-site, but the fender is completely out of service and the dock unprotected until repairs can be effected.

The foam-filled fender is unsinkable; when punctured it will remain afloat and operative until removed for repair. Foam-filled and pneumatic fenders have few maintenance problems. Aside from repair of accidental damage, the only maintenance required is occasional check of the fenders themselves and their supporting chains or cables.

Gravity fenders are normally made of concrete blocks and are suspended from heavily constructed wharf decks. Impact energy is absorbed by moving and lifting the heavy concrete blocks. High-energy absorption is achieved through long travel of the weights. Movement may be accomplished by a system of cables and sheaves, a pendulum, trunnions, or by an inclined plane. The main disadvantage of this system is the high initial cost and the high maintenance cost.

The "Dashpot" hydraulic fender system consists of a cylinder full of oil or other fluid so arranged that when a plunger is depressed by impact, the fluid is displaced through a non-variable or variable orifice into a reservoir at higher elevation. When ship impact is released, the high pressure inside the cylinder forces the plunger back to its original position and the fluid flows back into the cylinder by gravity. This system is most commonly used where severe wind, wave, swell, and current conditions exist. Its main disadvantages are high initial cost and high maintenance and repair cost.

The hydraulic-pneumatic floating fender consists of a floating rubber envelop filled with water or water and air, which absorbs energy by viscous resistance and/or by air compression. This fender seems to meet certain requirements of the ideal fender, but is considered to be expensive in combined initial and maintenance cost.

The steel spring fendering system is self-explanatory. Its main disadvantage is the corrosion characteristics of the steel.

FENDERING MATERIALS

The several types of fender systems described utilize timber in conjunction with other materials, depending on the design. The timber in each case is intended to absorb a certain amount of impact energy from docking ships or impact of collision and to function as a rubbing surface between the ships and dock.

Accordingly, the timber selected for fender use should have a relatively high compressive strength perpendicular to the grain to resist crushing action. Also, the wood should have a relatively high fiber hardness to resist the rubbing action, although this hardness should not be so great as to result in brittleness and checking in some fender systems, the timbers are often subject to sizeable bending stresses in which case the bending strength of the wood should, of course, be relatively high.

Aside from the structural strength requirements of the timber, the matter of existence of marine borers in a particular area should be considered as an important factor. Very severe marine borer activity necessitates the use of treated woods. Several agencies may be helpful in the structural capacity and preservation of wood.

1. Forest Products Laboratory at Madison, Wisconsin
2. Foreign Shipbuilding Woods Task Committee of the National Security Industrial Association of London, England
3. American Wood Preservers Institute, McLean, Virginia

Another material of importance in fendering systems is concrete used in gravity fenders. The materials utilized in the making of concrete should be of such quality as to provide sufficient strength which would last an indefinite period of time. Usually the specifications for gravity fenders specify 3,000 to 5,000 pounds per square inch (psi) concrete. The concrete should be made with a high sulfate resistance cement and should be protected against salt scaling. Agencies to consult are:

1. American Concrete Institute, Detroit, Michigan
2. American Society for Testing and Materials, Philadelphia, Pennsylvania
3. Cement and Concrete Research of Pennsylvania State University
4. Portland Cement Association, Skokie, Illinois
5. Prestressed Concrete Institute, Chicago, Illinois
6. American Society of Civil Engineers, New York, New York

Another important fender material is rubber. Most fendering systems utilize rubber in one way or another. As evidence of such, many companies manufacture rubber fenders or some form of it:

1. Seiba Rubber Company, Tokyo, Japan
2. Lord Manufacturing Corp.
3. General Tire and Rubber Co.
4. Uniroyal Tire and Rubber Co.
5. Yokahoma Rubber Corp, Houston, Texas
6. Bridgestone Rubber Corp., Rotherdam, Denmark

The final material used to some extend for fendering systems is structural steel. The structural steel used as reinforcement or in steel springs is of importance due to its corrosive properties. The structural grade and durability varies considerably and when utilized for fendering systems these values should be known prior to induction in the fendering system. Sources of information may be obtained from the American Iron and Steel Institute in Washington, DC, and the American Society of Civil Engineers in New York, New York.

COMPUTER PROGRAM

Because fendering systems utilize many design parameters and the calculations are long and tedious, a computer program was written to handle the basic seven types of fendering schemes and their classifications. Thus, the computer program has the ability to assist design and/or analyze any of the above mentioned fendering systems.

In the past, the solution of either the fendering system or dolphin has been examined by the engineer as a single element, fixed at the base (cantilever) and then applied as a basic physics relationship. The method rewards the engineer with simplicity but inherently may not be conservative or safe. This condition has led to the impetus of developing a computer-oriented solution of such systems. This can incorporate many variables, which are not possible in the simplified technique, and thus provide rapid and accurate solutions to a complex problem.

Ten assumptions are necessary to utilize the existing program; they are:

1. The piling interaction with the soil medium is considered, i.e., flexible supports.
2. The soil may be layered.
3. Piling group is considered as a three dimensional unit.
4. Interactions of the horizontal walers are considered.
5. Forces and deformations throughout all piles, at any time interval, can be evaluated.

6. The forces and deformations are evaluated along the length of each pile.
7. Rigid wharves, fenders, dolphins, or combinations can be considered.
8. During vessel impact, any pile that fails is noted and the system is reevaluated.
9. Total energy in the system, input and output, is computed during each time interval.
10. The system may have any general plan orientation, i.e., straight, curved, etc.

The computer program utilized was written on the UNIVAC 1108 computer and in the Fortran IV language. The basic theory utilized in a protective device system consists of several sub-systems. One sub-system consists of complete interaction of the supporting piling system, which includes any number of piles, pile types, and soil characteristics.

The other sub-system is the interaction of the system, supports, fenders (if applicable) and any distribution beams. This entire system is then examined under the impact of the vessel, at any attached angle. At any distance, the piling is examined for a failure mode. When a given pile fails, the system is automatically modified and the dynamic analysis is continued. Automatically this process is continued until the vessel stops or all the energy is consumed, i.e., failure of all piles. At each instant of a pile failure, the resulting forces and stress on this failed pile is listed.

Input consists of the size (tonnage), contours, speed and direction of approach of the vessel, rigidity and energy absorbing characteristics of the protective system and of the vessel, the soil parameters, and finally the geometry and size of the protective system and the materials used.

Output includes the velocity of the vessel at any instance and the load deformation of the protection. Further, it gives the energy absorbed by the protective system and the vessels at any distance.

The results are then interpreted as to whether the protective system is adequate for the given conditions or whether it is over or under designed. If the proposed protective system is found to be under designed for the given conditions, strengthening may forestall a major catastrophic failure. Recommendations can be made as to what structural elements to increase in size.

If the results are interpreted to be over designed, then recommendations can be made to decrease the size of the structural elements. In either the under or over designed case, dollars are saved, lawsuits are avoided (under designed) and materials may be saved (over designed).

STANDARDS AND CRITERIA

Protective systems such as fenders, dolphins, cells and platforms contribute to navigation safety by minimizing or preventing damage to the bridge, pier or vessel. They may protect the marine environment by minimizing spills from punctures. Because of arbitrary limitations placed on design of protective systems many are underdesigned. Since it is improbable that designers and contractors have appropriate computer programs design standards and criteria are needed.

The American Railway Engineering Association (AREA) has recently issued a set of design and computer standards titled "Pier Protection Systems at Spans Over Navigable Streams." These Standards are indeed helpful but rely mostly on a state-of-the-art approach. They fail to take into account that vessels are increasing in size and that the old protective systems and methods of design are no longer adequate.

The American Association of State Highway and Transportation Officials (AASHTO) have no standards dealing with bridge and pier protective systems and devices. Because no standards exist for AASHTO and standards of AREA are inadequate, the author proposes the following:

1. Bridge Pier Protection

A. Definition

In order to assure adequate protection against vessel collision, combinations of piling, dolphins and fenders must be provided. These units should be designed to reduce the vessel's velocity and possibly redirect the vessel to avoid contact with the piers.

B. Vessel Dimensions

The following average vessel dimensions, Table 1-5, may be used in design and layout of bridge and pier protective systems and devices.

2. Vessel Velocities

The velocity of impact in feet per second (1.5 fps is 1 mile per hour) of the vessels can be selected from the following:

Condition (Wind and Swell)	Approach	Ship Displacement Up To 3,000 Ton	Up To 10,000 Ton	Over 10,000 Tor
Strong	Difficult	2.5	2.0	1.1
Strong	Favorable	2.0	1.1	1.0
Moderate	Difficult	1.0	0.8	0.6

3. Horizontal Live Load

A. Force-Acceleration Method

The applied horizontal force to an individual pile, fender or dolphin is computed from:

$$P = K_e \; YC \qquad (1)$$

where: P = Applied Horizontal Force

$$K_e = \frac{K_p \cdot K_f}{K_p + K_f}$$

$K_p = (L_p^3/3\ El_p)$D.F.

K_f = Fender, stiffness $(0.1 \leq K_f \leq 60$, average of 30)
If $K_f = 0$ then $K_e = K_p$

L_p = Length of pile

E = Modulus of elasticity of the material

l_p = Moment of inertia of the pile

D.F. = Distribution Factor

$Y = V_o/\lambda$

V_o = Initial velocity of vessel (in/sec)

$\lambda = (K/M)^{\frac{1}{2}}$

$M = W_S/32.2(\text{ksec}^2/\text{in})$

W_S = Weight of the vessel

$$C = C_E \cdot C_C \cdot C_H$$

C_E = Eccentricity Coefficient, where C_E is determined from Fig. 9

C_C = Configuration Coefficient, where C_C equals

Pier Type	C_C
Open	1.0
Semi-Closed	0.9
Closed	0.8

C_H = Hydrodynamic Mass Coefficient - 1 + 2 D/B

D = Draft of vessel

B = Beam of vessel

The resulting acceleration and stopping time is computed from

$$a = V_o \lambda \qquad (2)$$

$$t = \pi/2\ \lambda \qquad (3)$$

Thus, with the use of the above equations any fender, dolphin, or protective system can be designed. The difference from normal design criteria (which has proved to be inadequate) is the use of the distribution factor. The following equation may be used to determine the D.F.

$$D.F. = [-6.0x10^{-7}\ D_x + F]\ L_p^{-0.006} \qquad (4)$$

where: $F = -3.5 \times 10^{-13}(D_y)^2 + 3.1 \times 10^{-7} D_y + 0.335$

$D_y = \frac{El_p}{S_p}$ (vertical pile stiffeners)

$D_x = \frac{El_\omega}{S_\omega}$ (transverse stiffeners of walers)

S_p = Spacing between piles (in.)

S_ω = Spacing between walter (in.)

The resulting maximum stress is computed from:

$$f = P \cdot L_p/S \qquad (5)$$

where: S = Section Modulus

If one wishes to disregard the distribution factor formula Figures 1-8 may be used.

TABLE 1: INLAND VESSELS

Designation	Horsepower	Carrying Capacity (Kips)	Length (ft)	Beam (ft)	Draft (ft)
TOWBOATS	1000-2000	-	117	30	7.6
	2000-4000	-	142	34	8.0
	4000-6000	-	160	40	8.6
TUGBOATS	350-650	-	65-80	21-23	8.0
	800-1200	-	90	24	10-11
	1200-3500	-	95-105	25-30	12-14
	2000-4500	-	125-150	30-34	14-15
OPEN HOPPER BARGES	-	2000	175	26	9
	-	3000	195	35	9
	-	6000	290	50	9
COVERED DRY CARGO BARGES	-	2000	175	26	9
	-	3000	195	35	9
LIQUID CARGO (TANK)BARGES	-	2000	175	26	9
	-	1500	195	35	9
	-	3000	290	50	9
DECK BARGES	-	700	110	26	6
	-	1800	130	30	7
	-	2400	195	35	8
CARFLOATS	-	-	257	40	10
	-	-	366	36	10
SCOWS	-	700	90	30	9
	-	2000	120	38	11
	-	2700	130	40	12

TABLE 2: REPRESENTATIVE CONTAINER SHIPS

(Seagoing Vessels)

Tonnage DWT (kips)	Displacement (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)	No. of Containers (circa)
112000	164640	951	106.3	42.7	2800
80530	114240	888	104.3	38.4	2000
56000	76160	696	98.4	35.1	1380
33600	44800	591	86.9	29.5	810
15680	21504	469	62.3	21.3	3.6

TABLE 3: SEAGOING FISHING VESSELS

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
5600	-	6272	295	45.9	19.4
4480	-	5600	279	42.7	18.4
3360	-	4704	262	39.4	17.4
2240	-	3920	246	36.1	16.4
1792	-	3472	230	34.4	15.7
1344	-	2688	213	32.8	14.8
896	-	1792	180	27.9	13.1
448	-	896	131	23.0	11.5

TABLE 3A: SEAGOING PASSENGER VESSELS

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Length between Perps (ft)	Beam (ft)	Draft (ft)
179200	-	16800	1033	968	116.5	37.7
156800	-	145600	1033	968	111.5	36.1
134400	-	123200	1017	951	106.6	34.4
112000	-	100800	984	919	101.7	34.4
89600	-	78400	869	804	96.8	32.8
67200	-	67200	755	689	91.9	32.8

TABLE 4: REPRESENTATIVE MIXED CARGO FREIGHTERS

(Full Deck Construction Seagoing Vessels)

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
22400	33600	44800	541	70.5	31.2
16800	24640	33600	492	65.6	29.5
11200	16800	22400	443	57.4	26.2
8960	13440	17920	394	52.5	24.6
6720	10080	13440	344	47.6	23.0
4480	6720	8960	312	42.7	19.7
3360	4928	6720	295	39.4	18.0
2240	3360	4480	246	34.4	14.8
1120	1568	2240	197	27.9	11.5

TABLE 5: REPRESENTATIVE BULK CARGO FREIGHTERS

(Ore, Oil, Coal, Grain, etc. Seagoing Vessels)

Tonnage (kips) GRT	Tonnage (kips) DWT	Displacements (kips)	Overall Length (ft)	Beam (ft)	Draft (ft)
-	2240000	2564800	1677	228.7	106.6
-	1568000	1803200	1545	259.2	95.1
-	1008000	1173760	1391	224.7	82.0
-	761600	896000	1306	205.1	75.5
-	504000	604800	1168	175.5	67.3
-	280000	347200	968	142.7	52.5
-	100800	134400	755	95.1	37.7
-	56000	67200	623	80.4	34.4

GRT - Gross Registered Tonnage

DWT - Dead Weight Tonnage

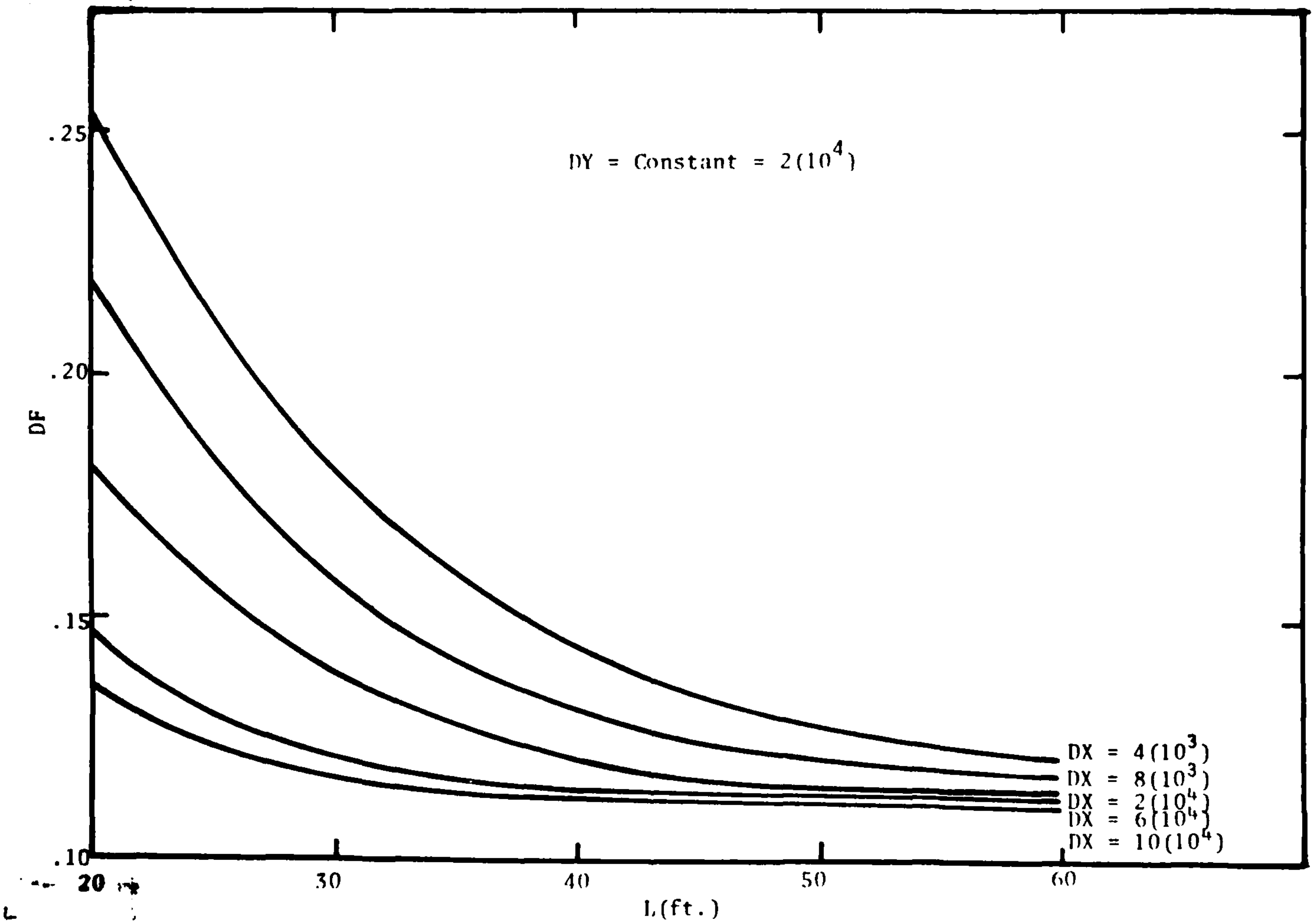

FIGURE 1. **Distribution Factor** $(D_y) = 2 \times 10^4$ k-in.

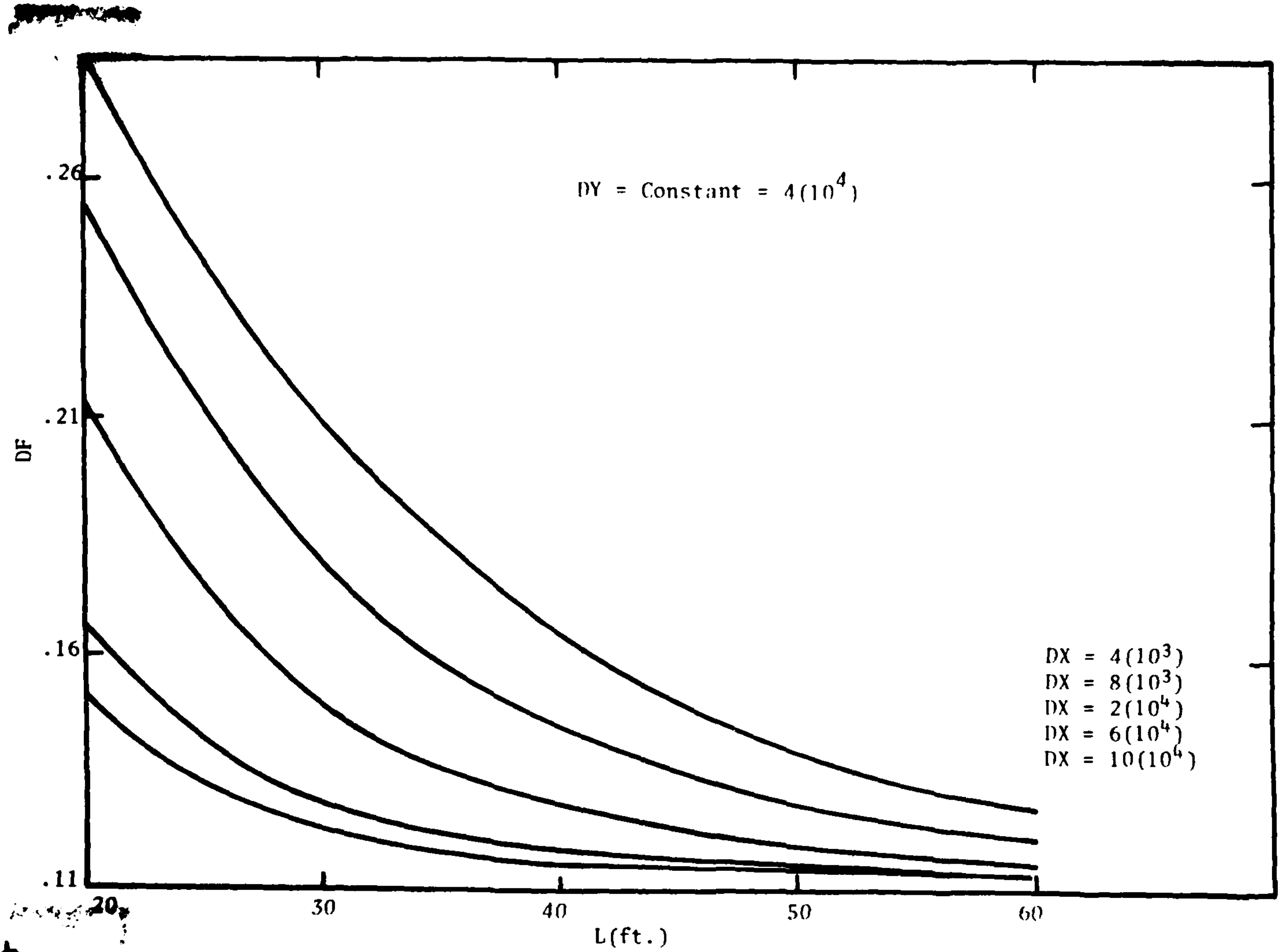

FIGURE 2. Distribution Factor $(D_y) = 4 \times 10^4$ k-in.

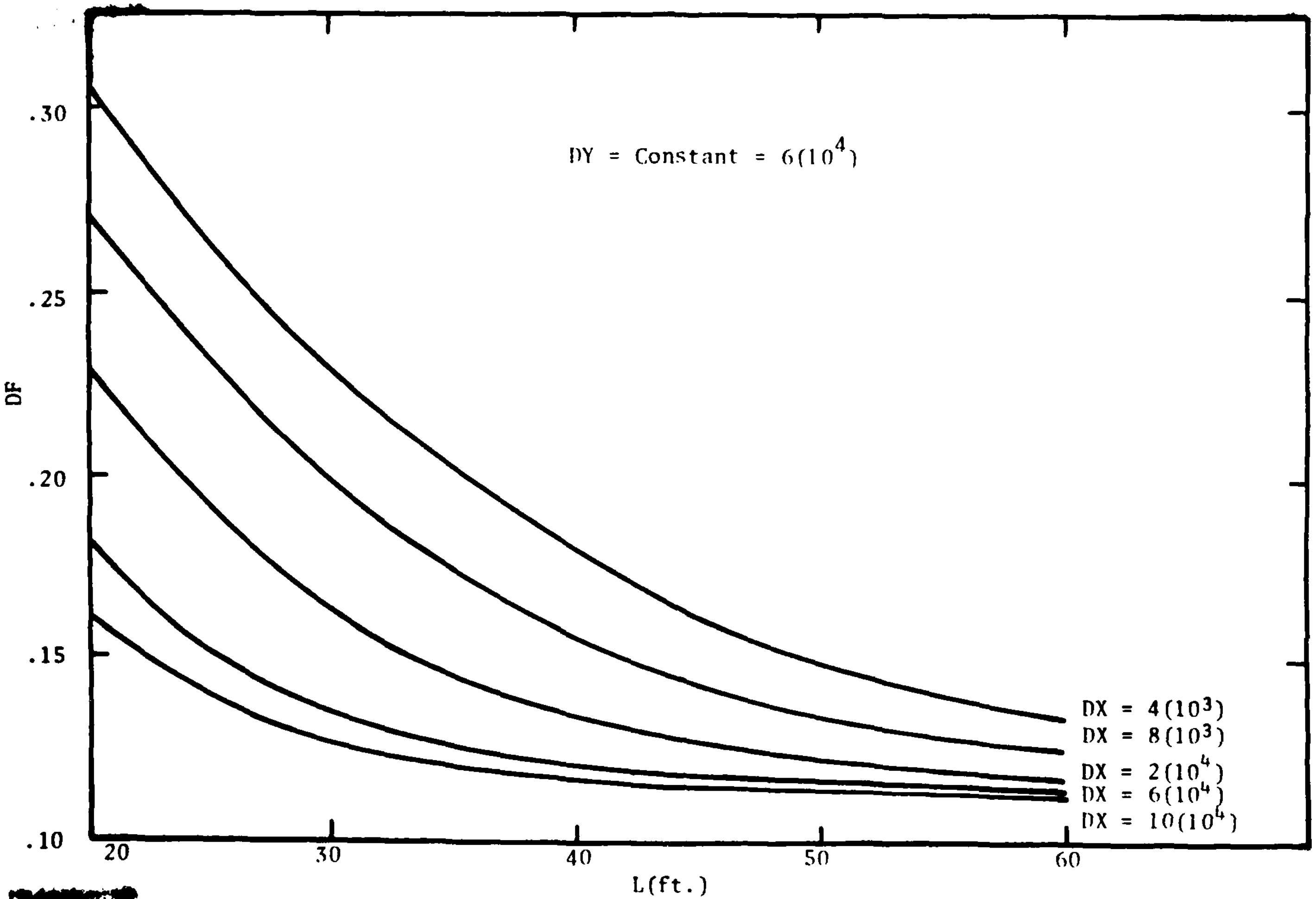

FIGURE 3. Distribution Factor (D_y) = 6×10^4 k-in.

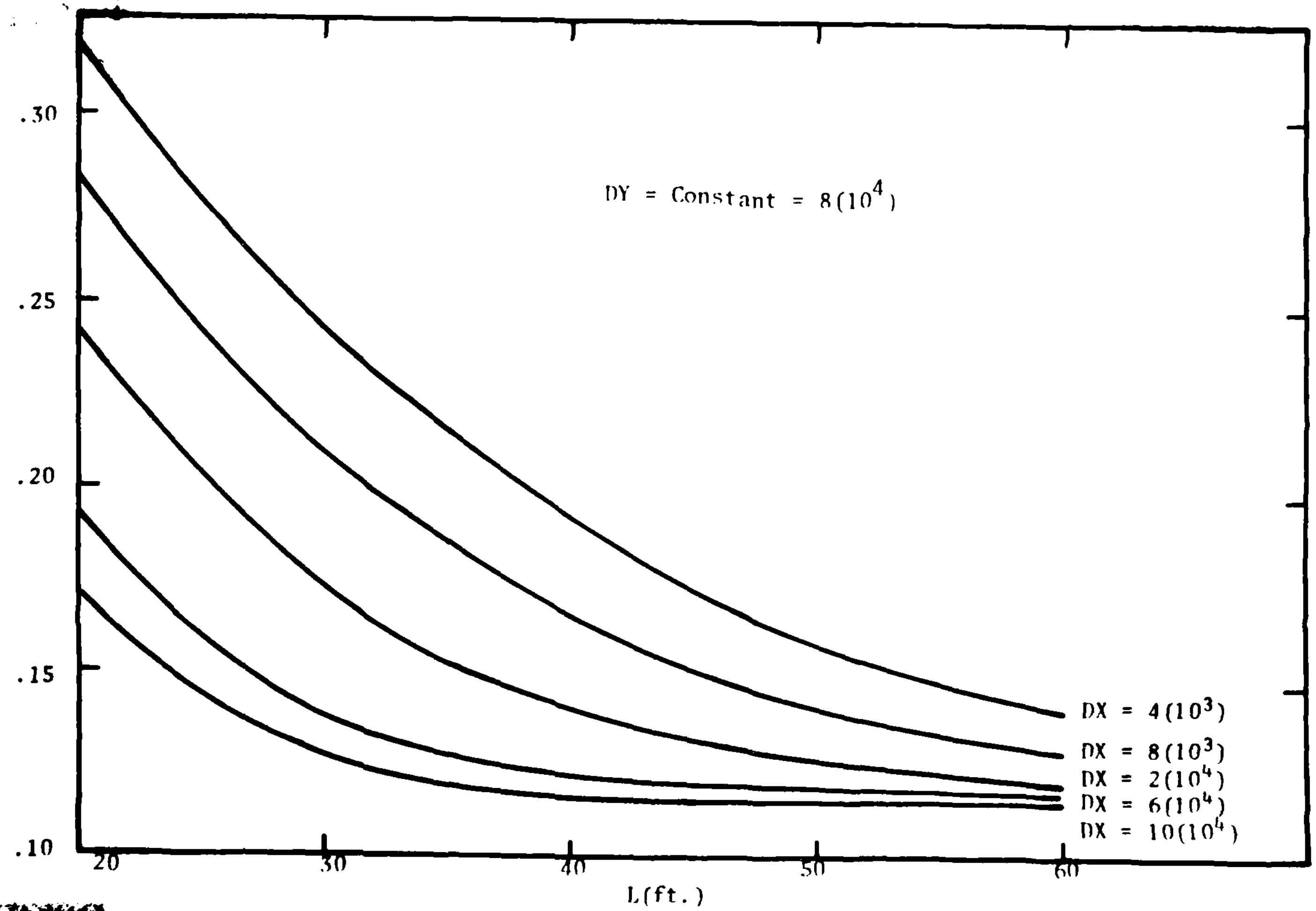

FIGURE 4. Distribution Factor (D_y) = 8×10^4 k-in.

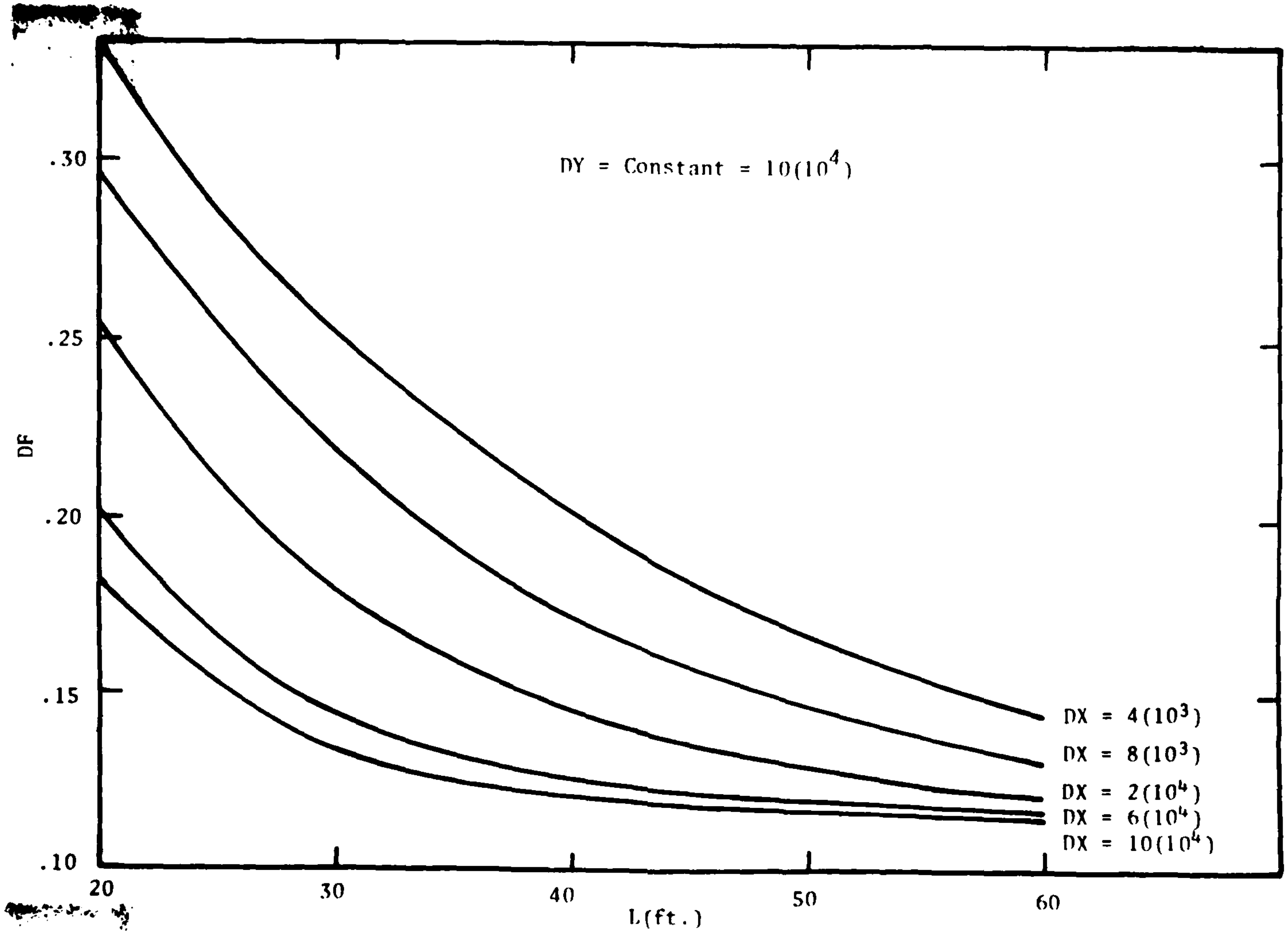

FIGURE 5. Distribution Factor $(D_y) = 10 \times 10^4$ k-in.

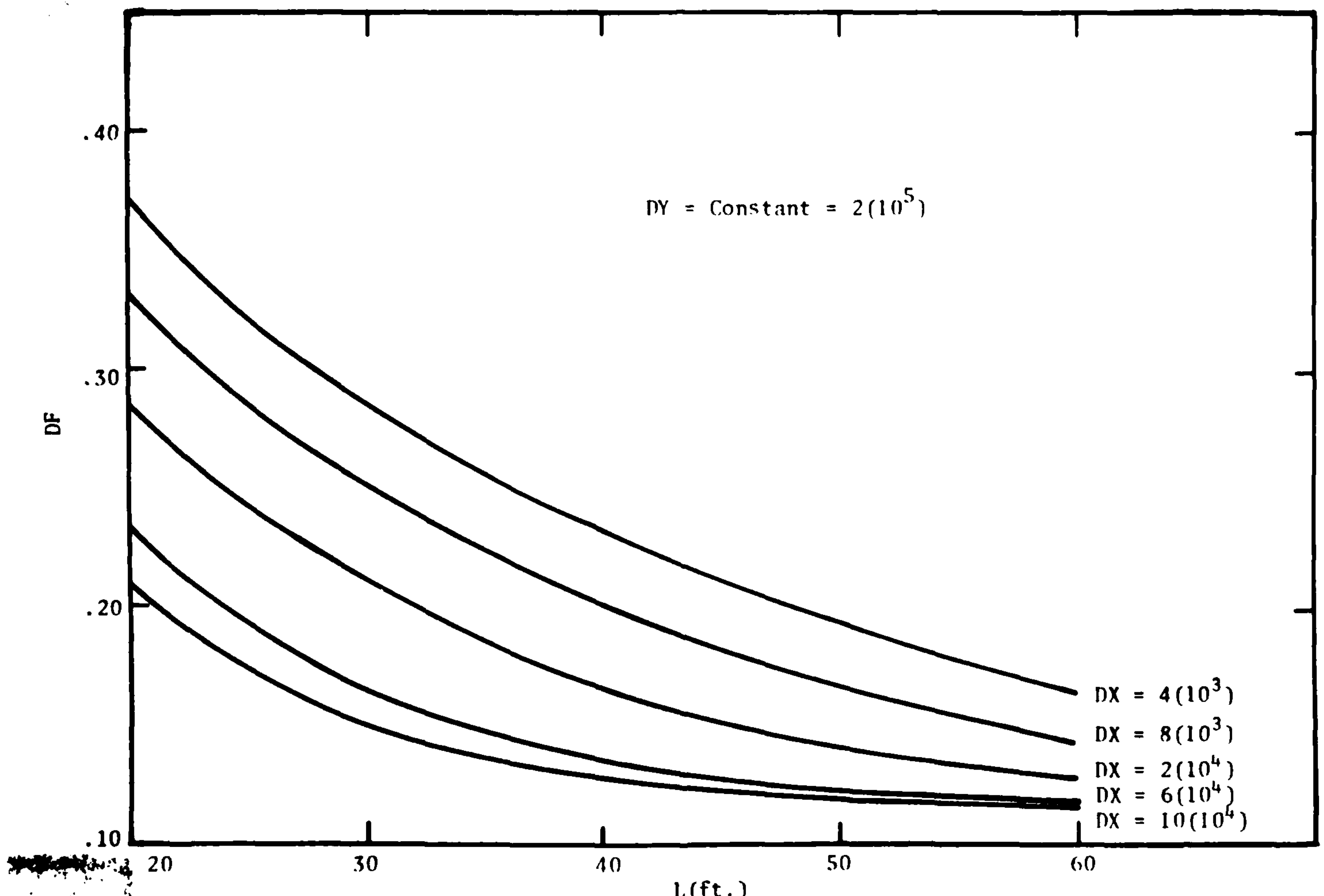

FIGURE 6. Distribution Factor (D_y) = 2 x 10^5 k-in.

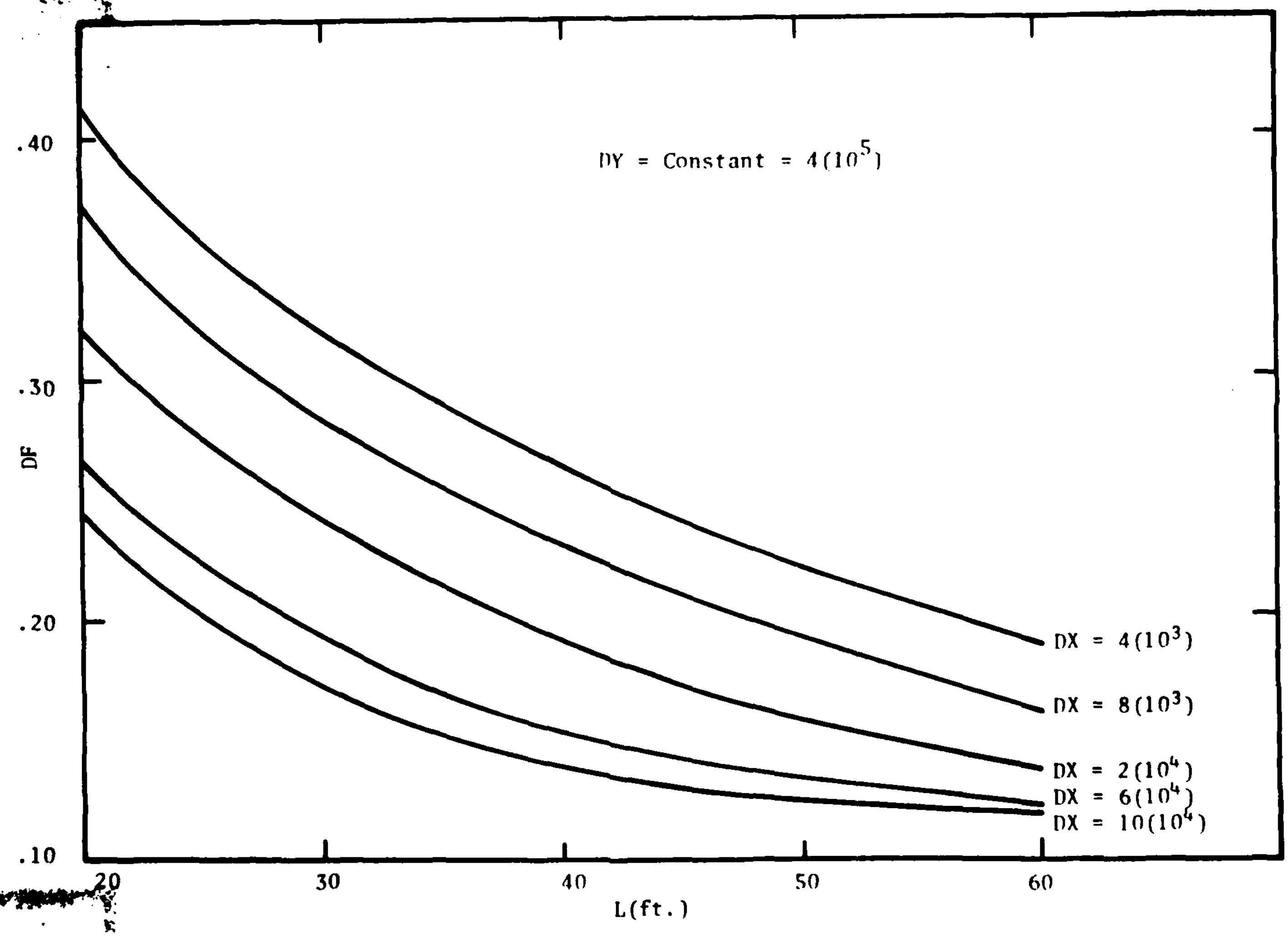

FIGURE 7. Distribution Factor (D_y) = 4×10^5 k-in.

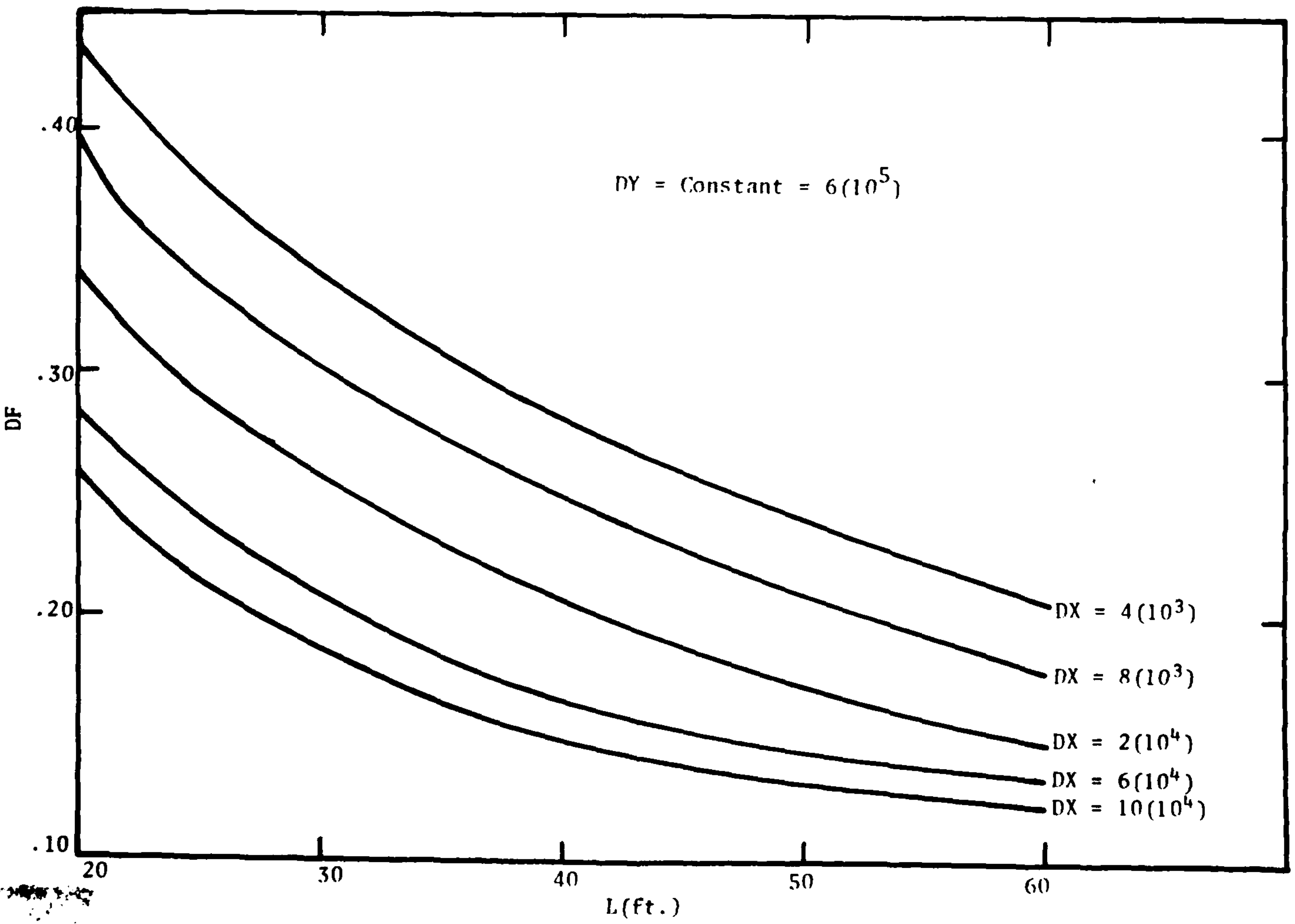

FIGURE 8. Distribution Factor $(D_y) = 6 \times 10^5$ k-in.

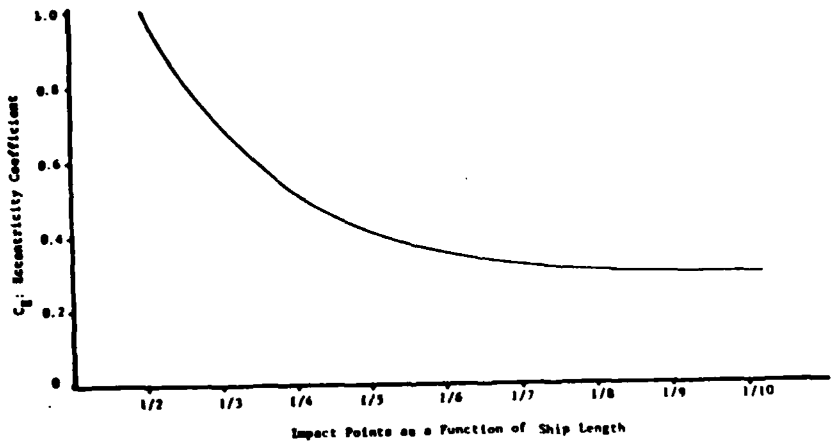

Fig 9: Eccentricity Coefficient vs. Impact Points

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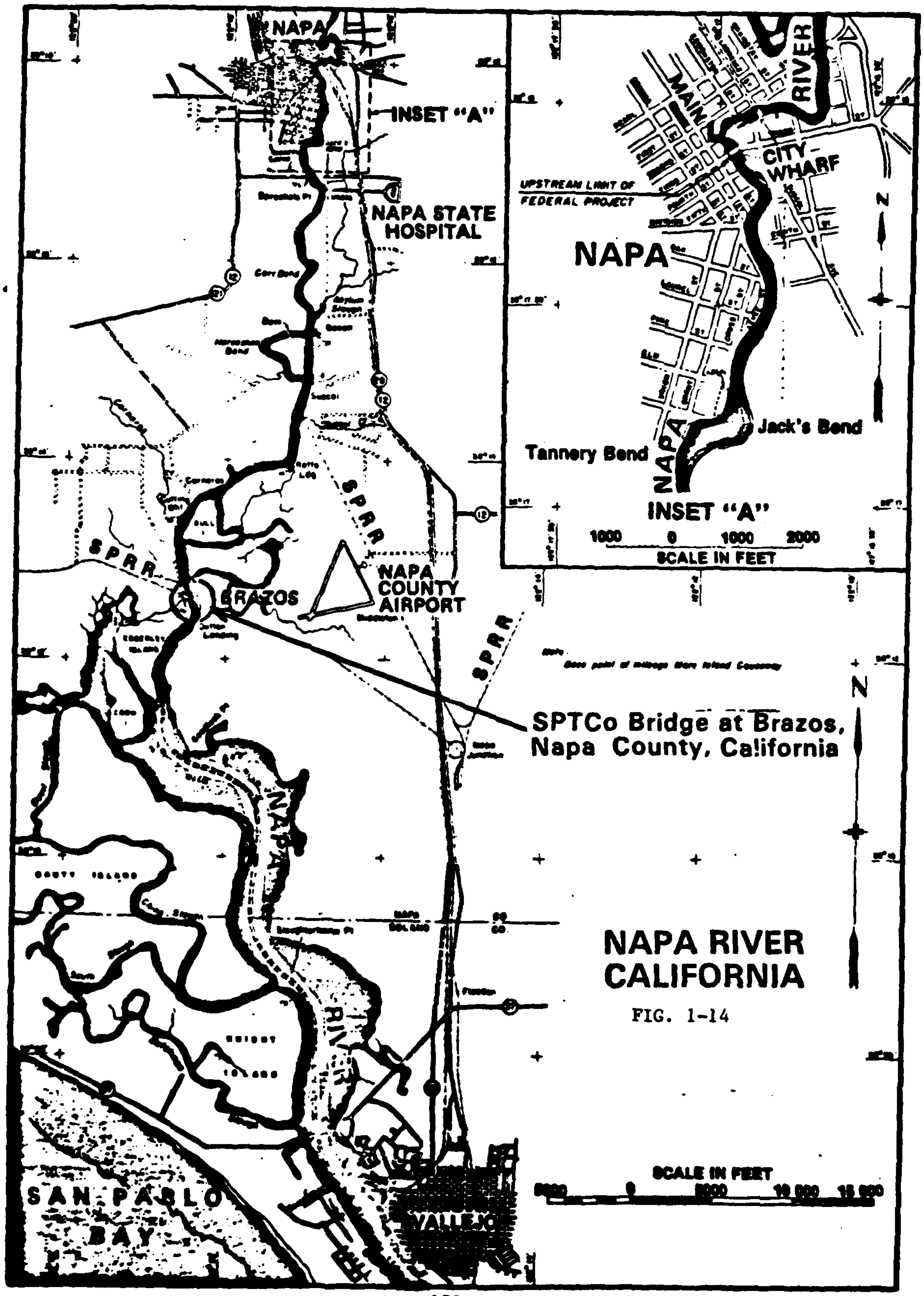
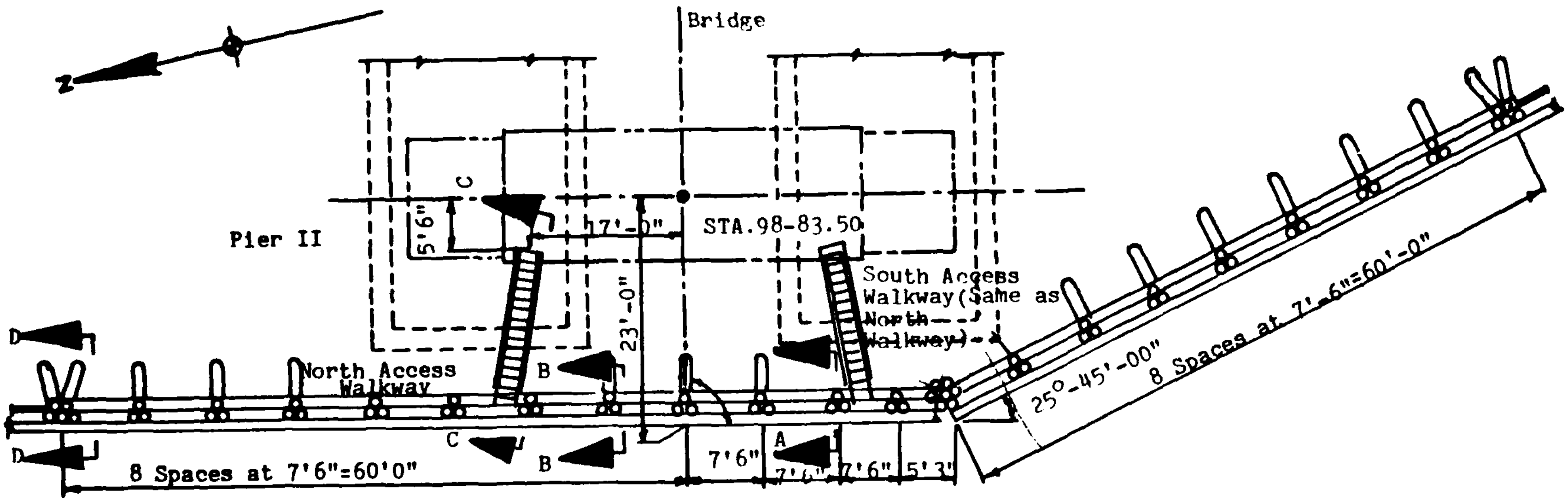
APPENDIX A

NAPA RIVER BRIDGE



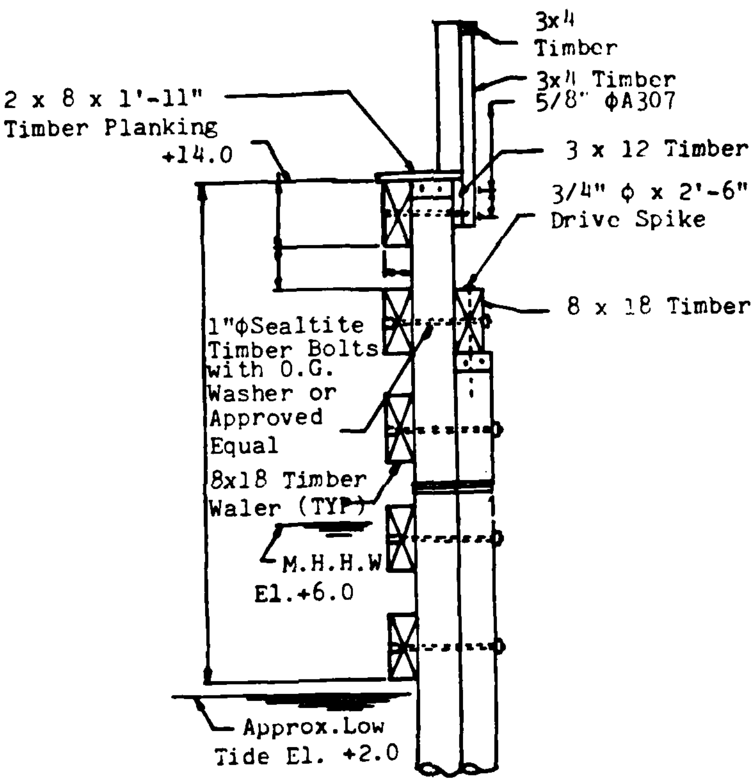
NAPA RIVER
CALIFORNIA

FIG. 1-14



FENDERING PLAN

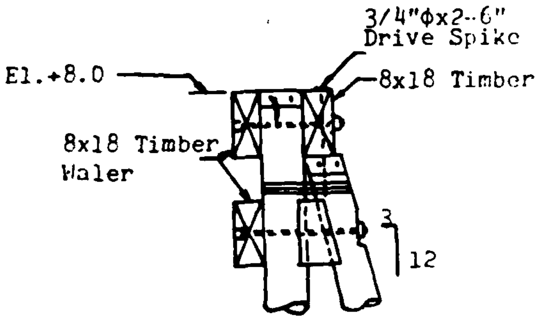
FIG. 1-15



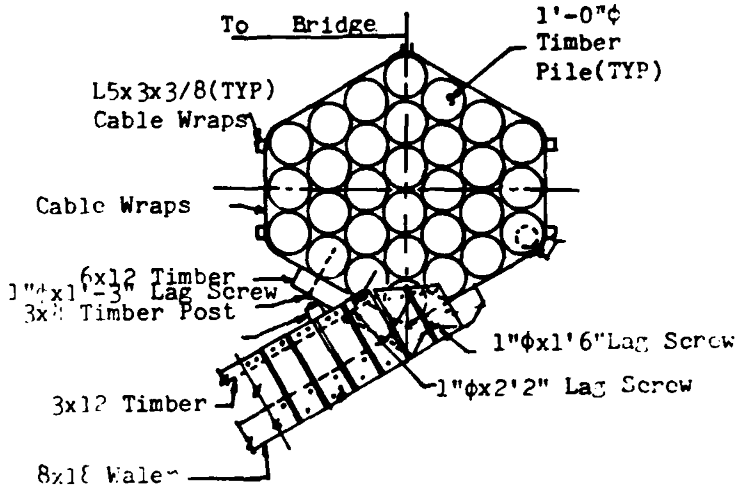
SECTION A-A

FIG. 1-16

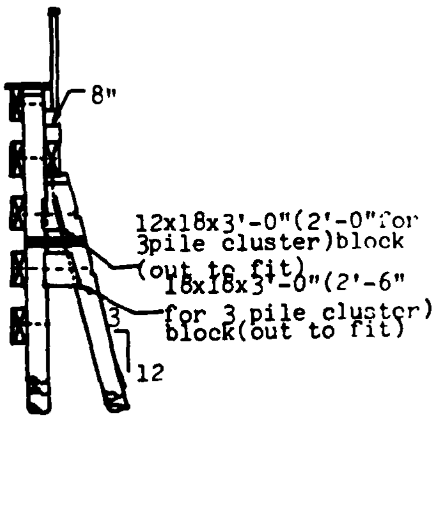
34



SECTION B-B



SECTION C-C

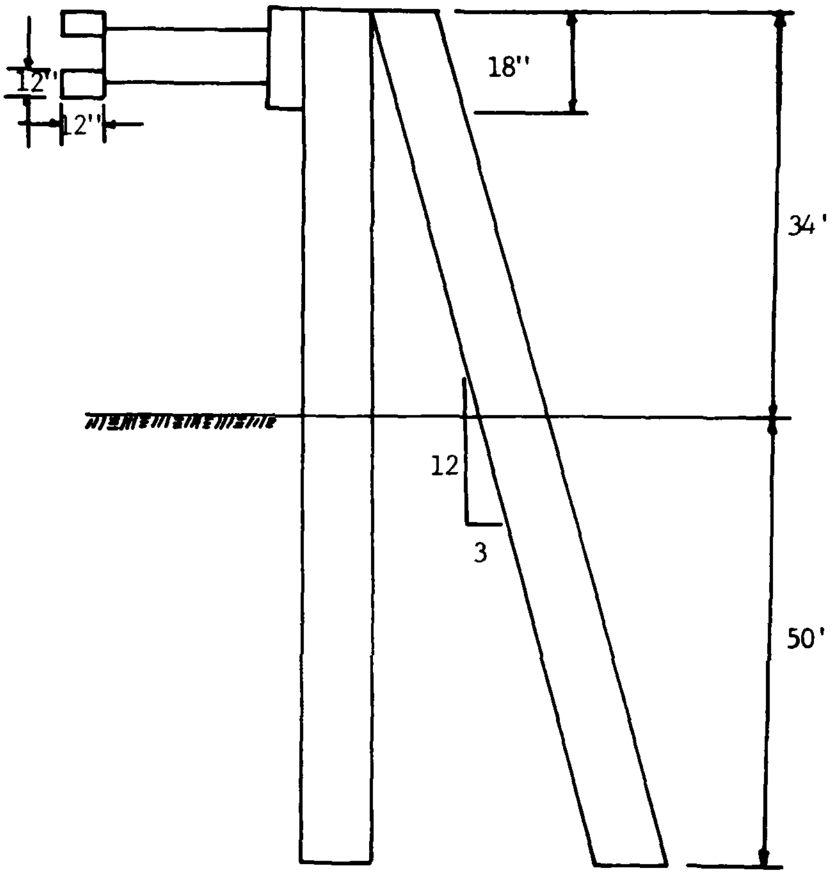


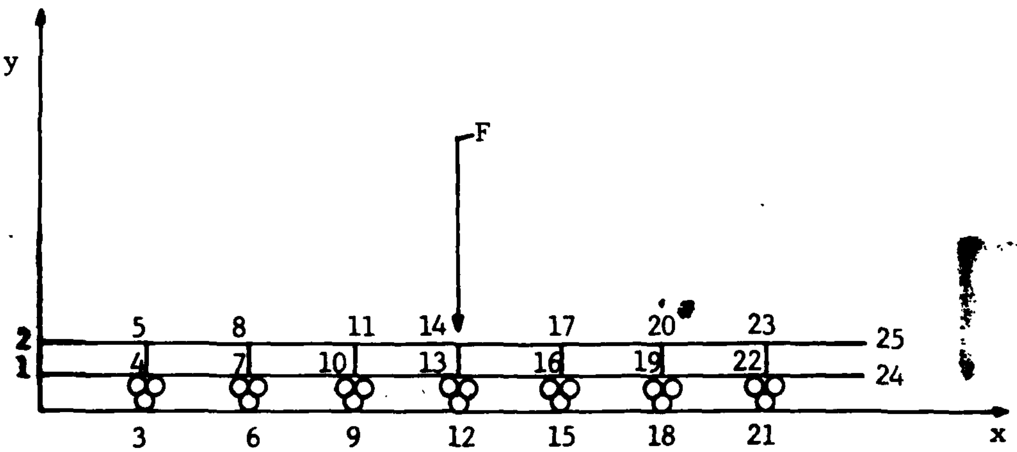
SECTION D-D

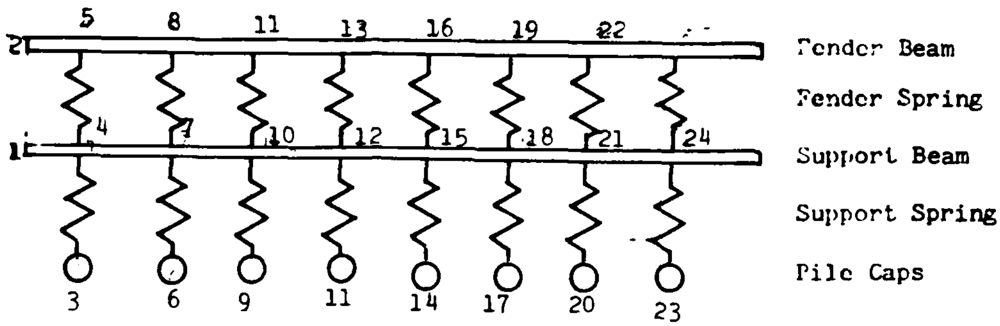
FIG. 1-17

NAPA RIVER BRIDGE

FIG. 1-18







FENDERING MODEL

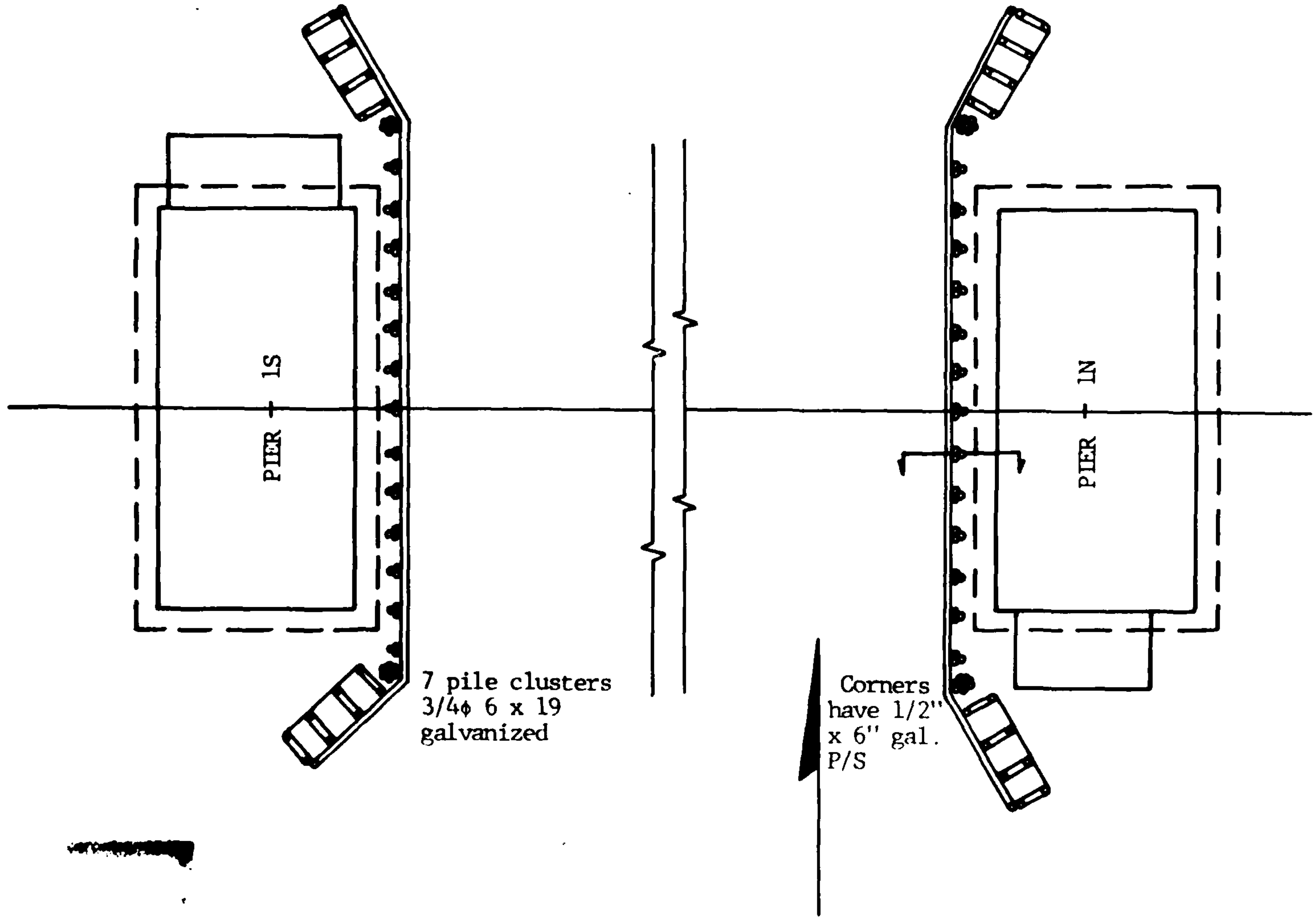
FIG. 1-19

DOLPHIN MODELS

THIRD STREET BRIDGE

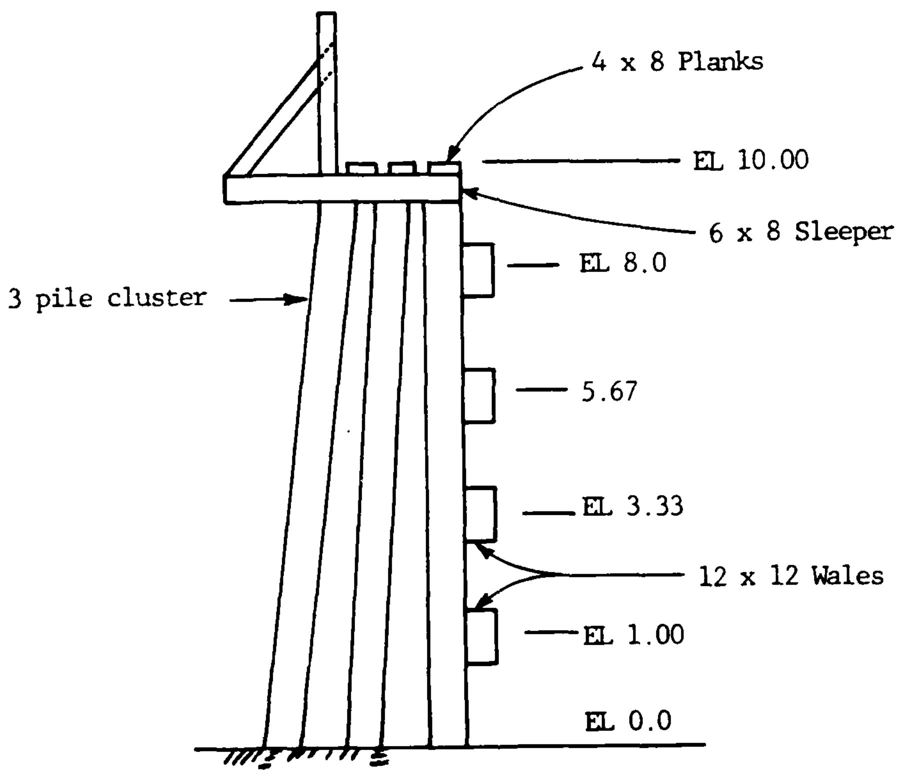
THIRD STREET BRIDGE

FIG. 1-20



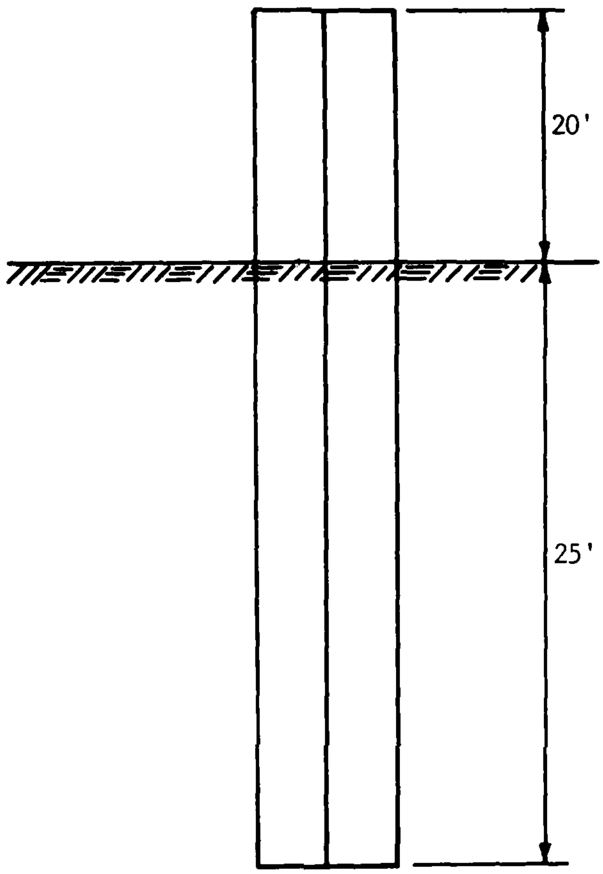
THIRD STREET BRIDGE

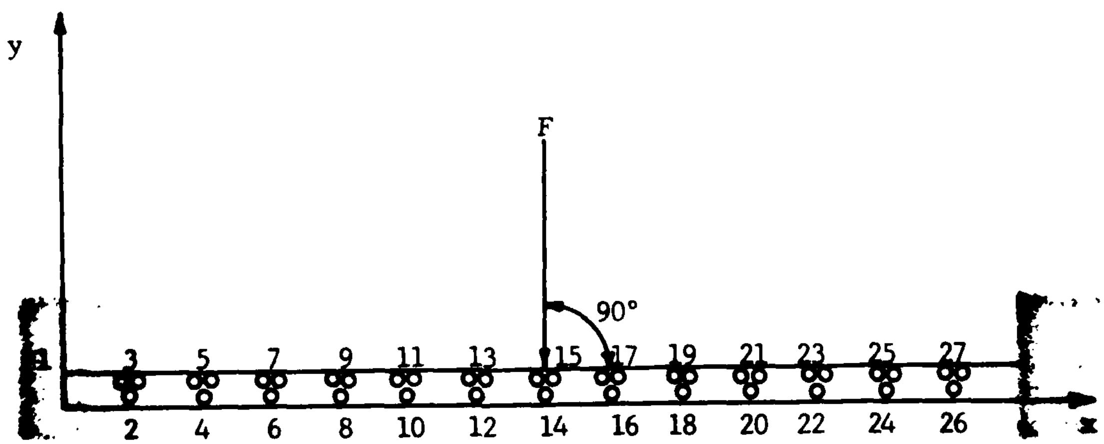
FIG. 1-21



THIRD STREET BRIDGE

FIG. 1-22

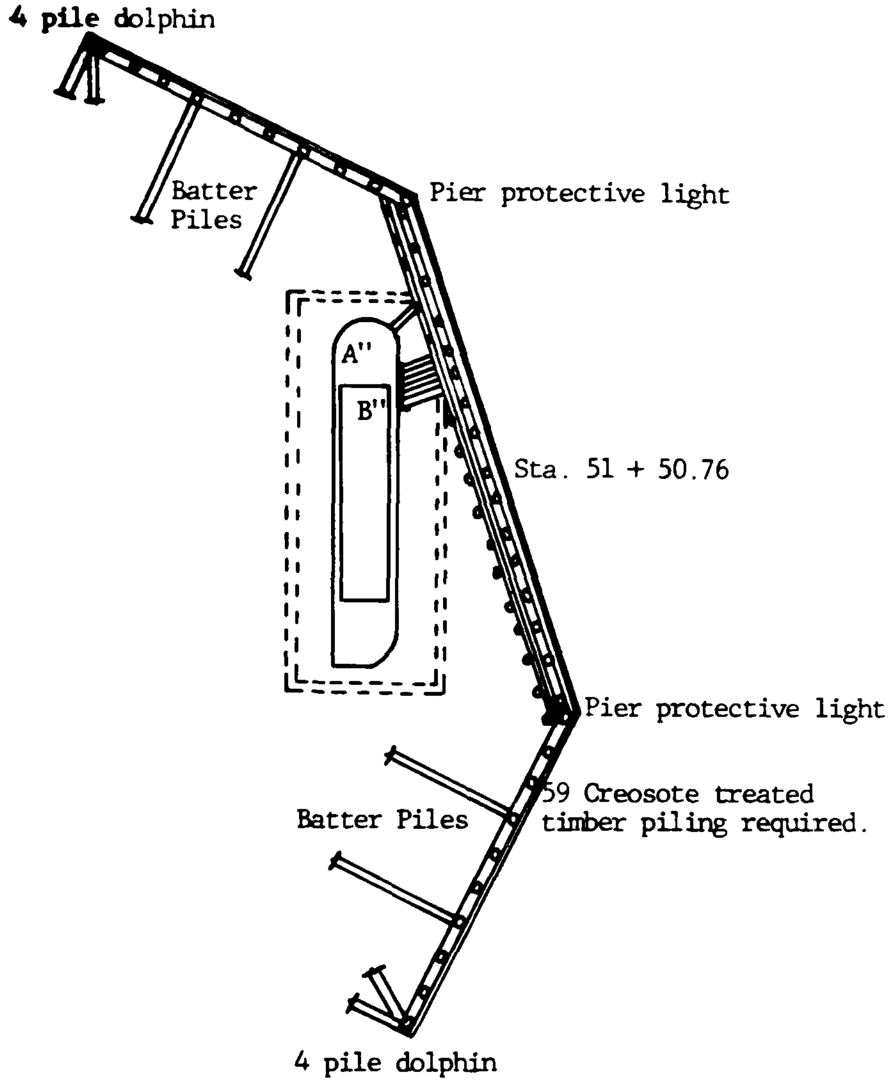




STEAMBOAT SLOUGH BRIDGE

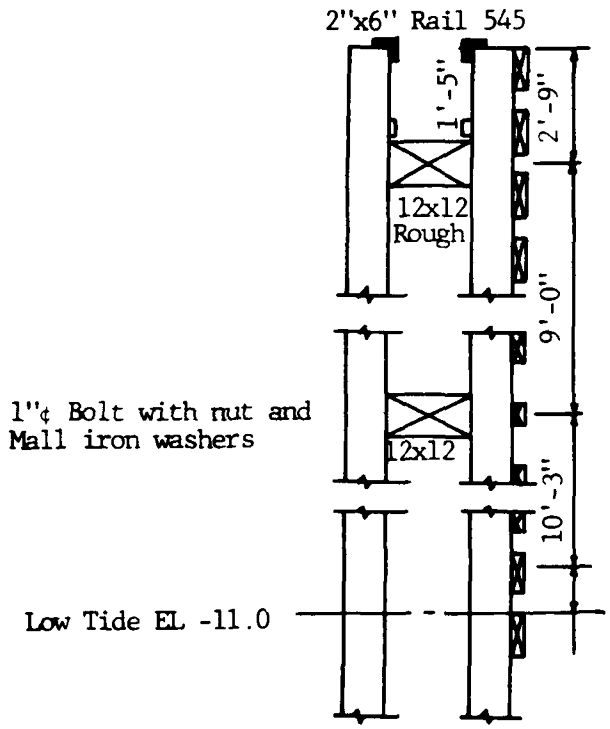
STEAMBOAT SLOUGH BRIDGE

FIG. 1-23



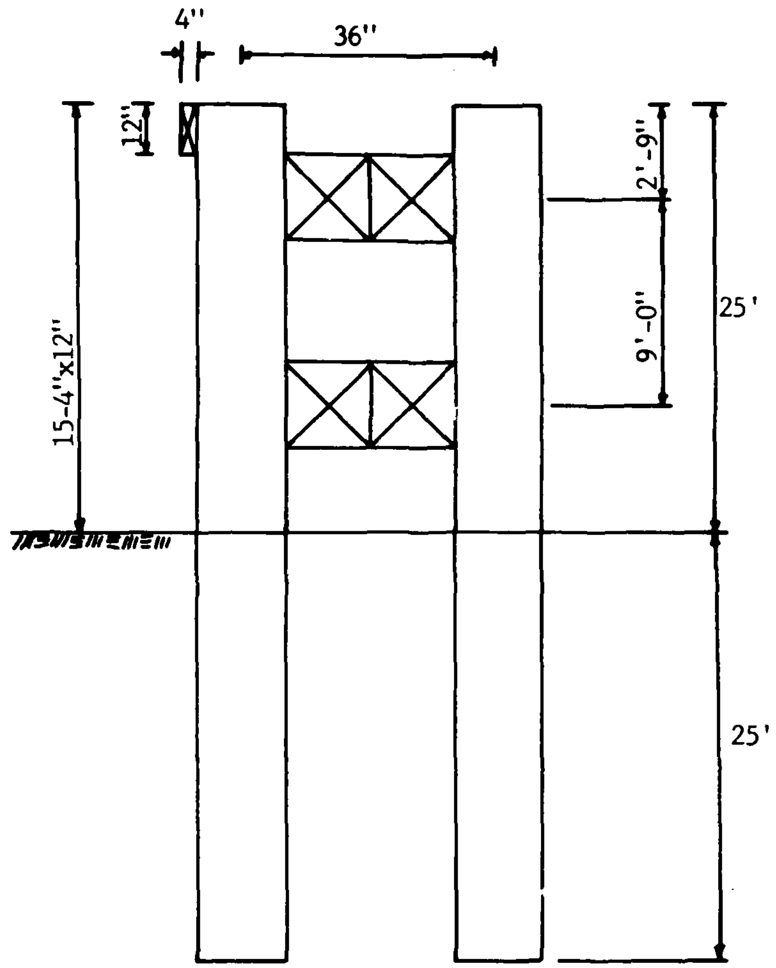
STEAMBOAT SLOUGH BRIDGE

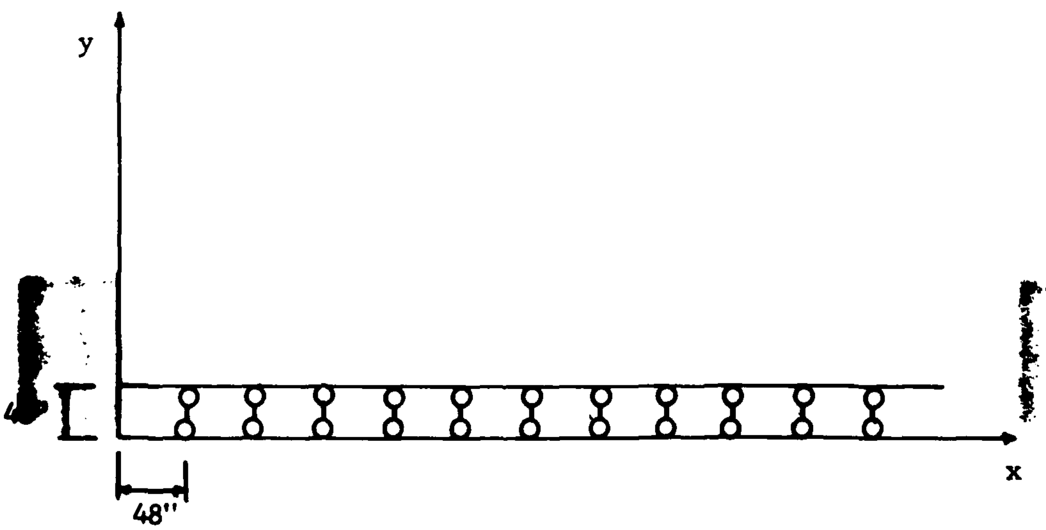
FIG. 1-24



STEAMBOAT SLOUGH RIVER BRIDGE

FIG. 1-25

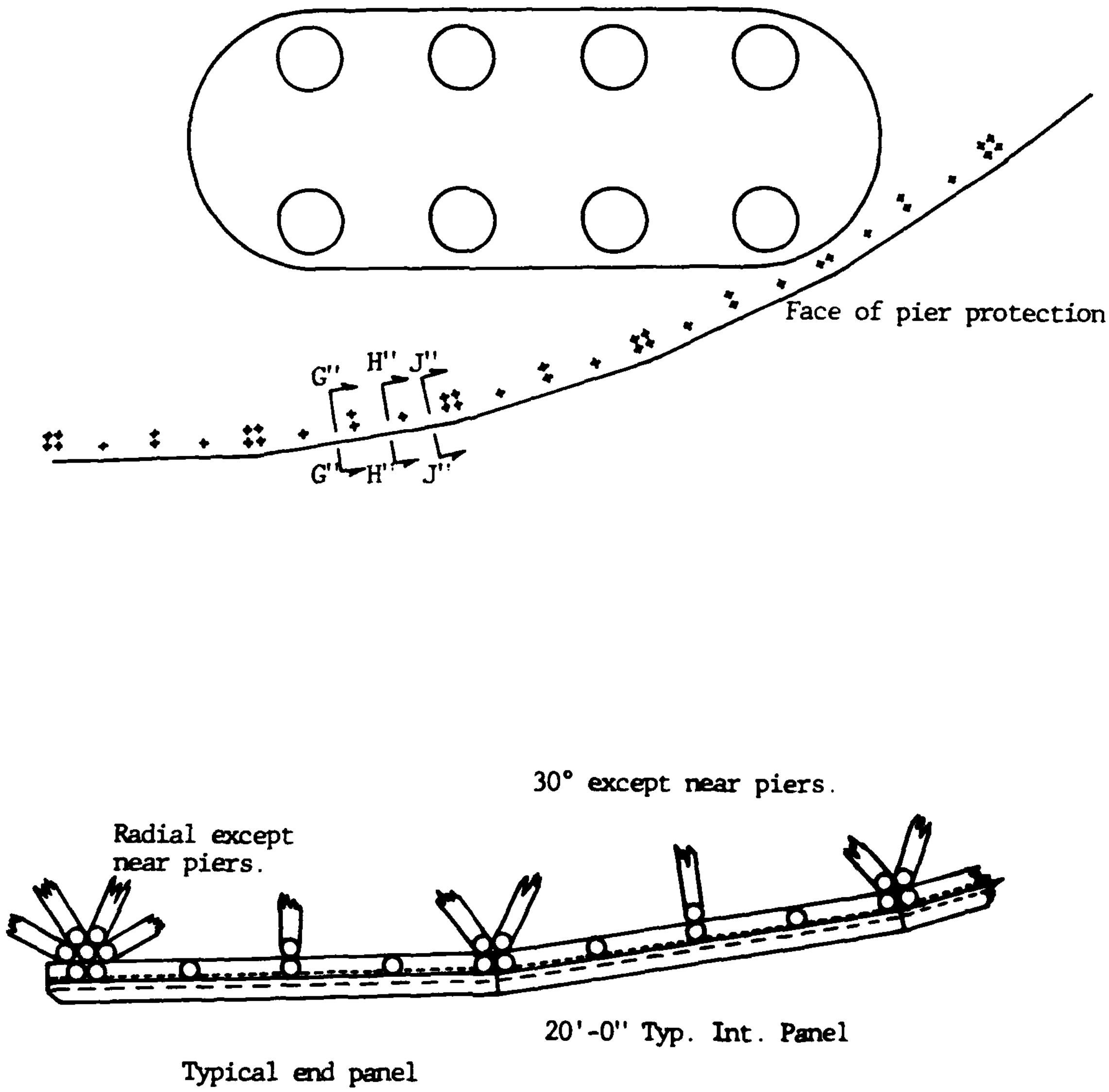




HOQUIAM RIVER BRIDGE

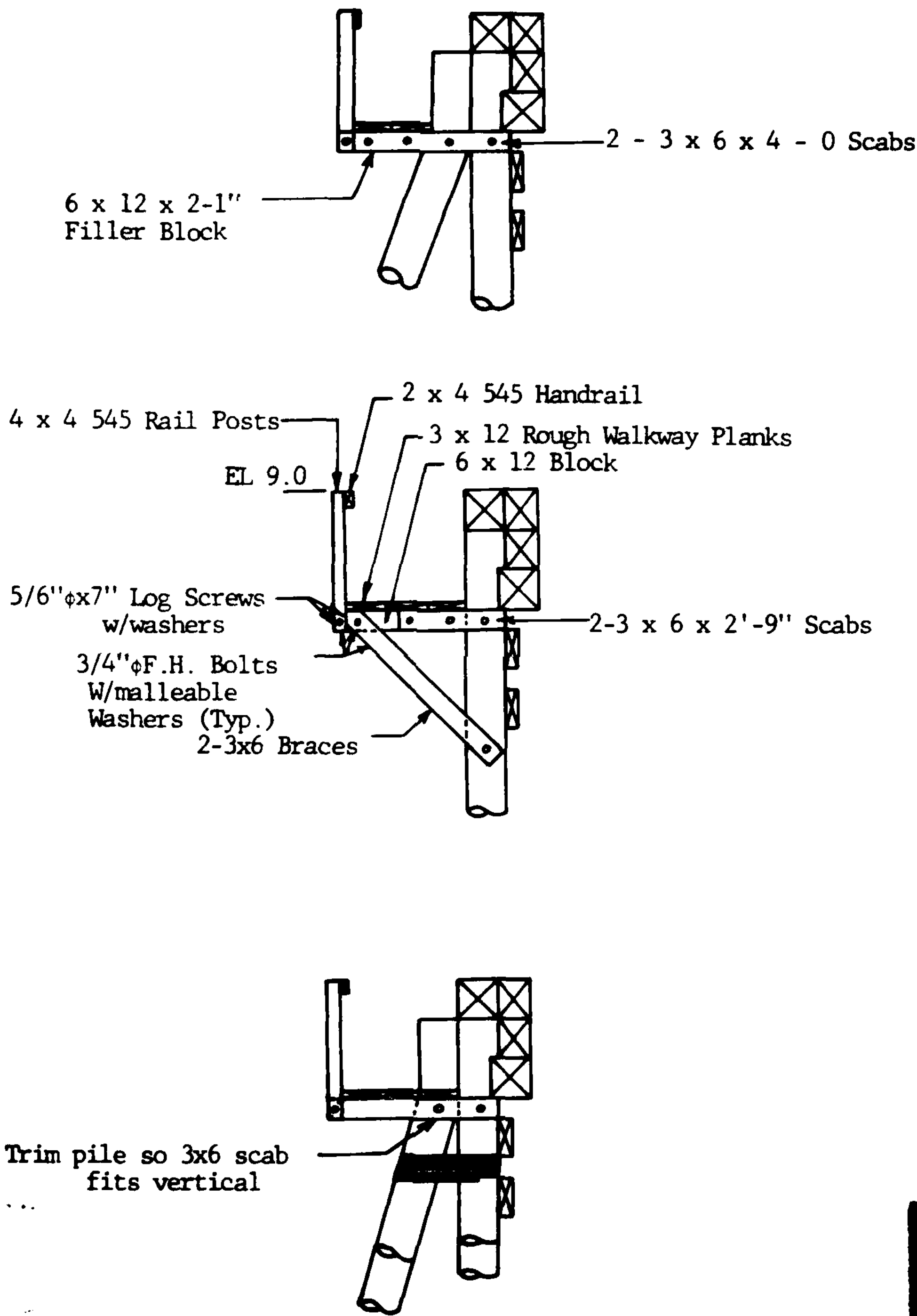
HOQUIAM RIVER BRIDGE

FIG. 1-26



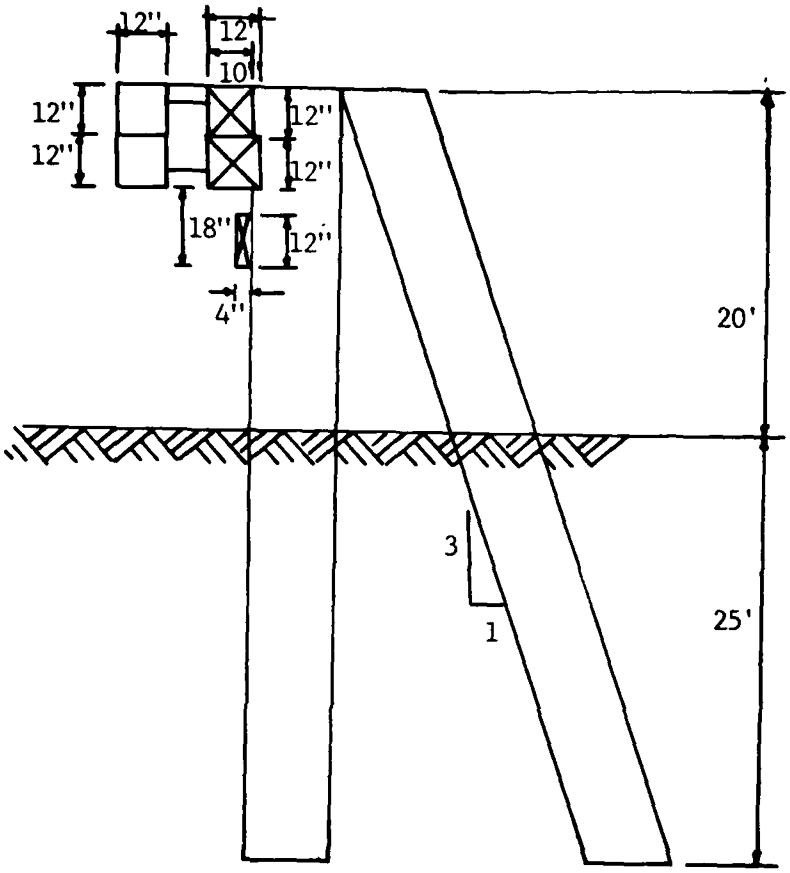
HOQUIAM RIVER BRIDGE

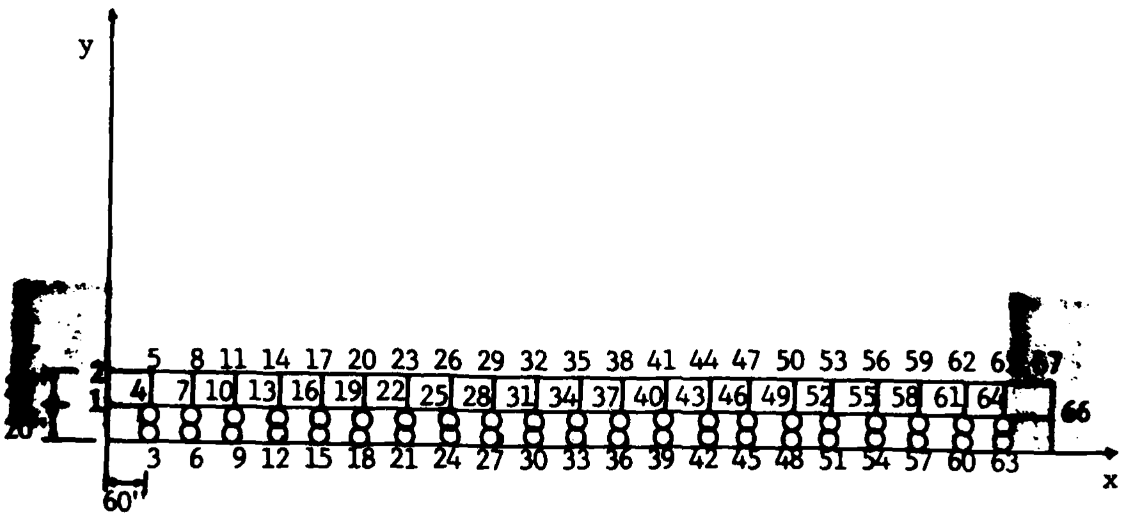
FIG. 1-27



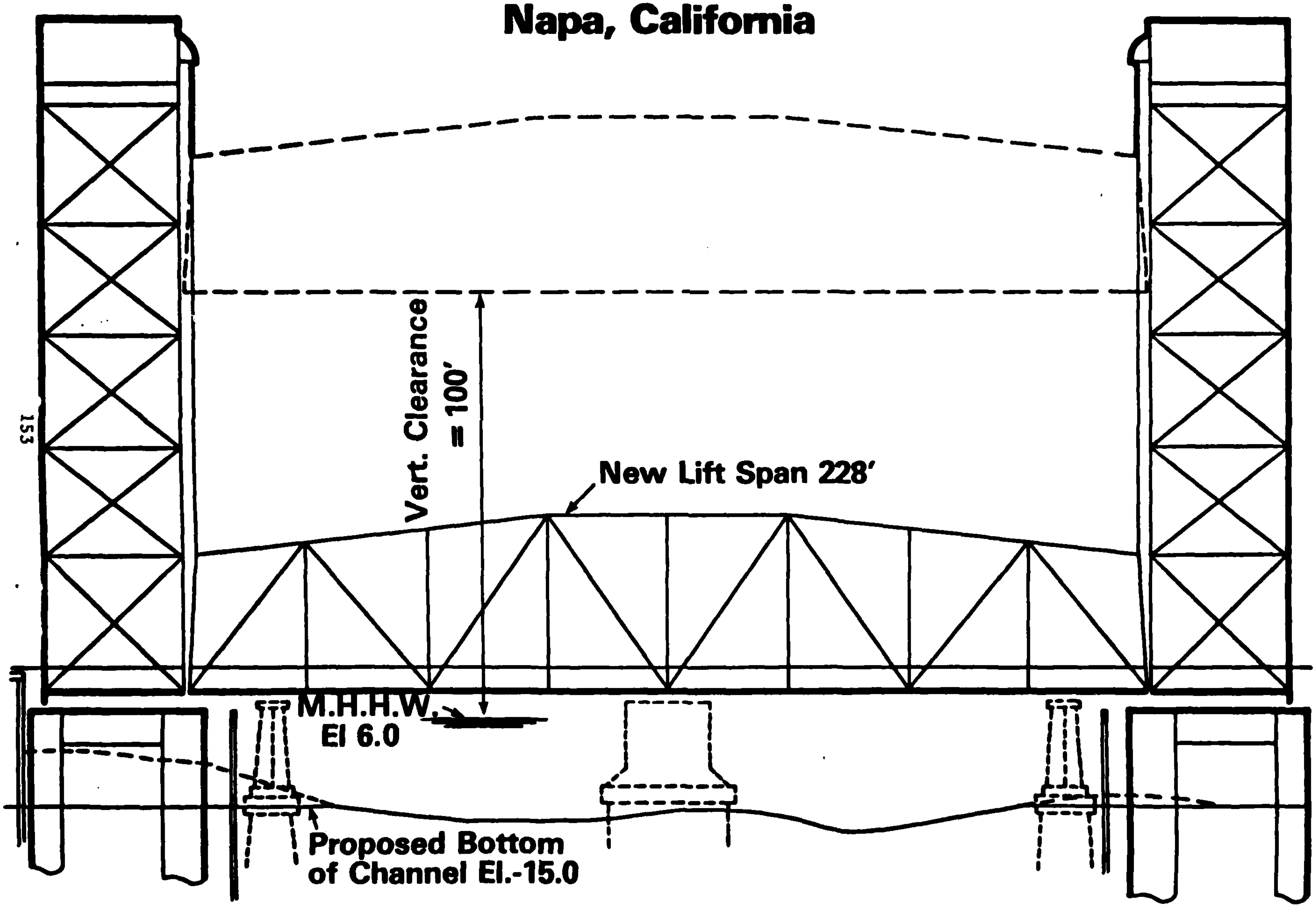
HOQUIAM RIVER BRIDGE

FIG. 1-28





Alteration of the SPTCO Bridge
Napa, California



SCHUYKILL RIVER BRIDGE

FIG. 1-30 A

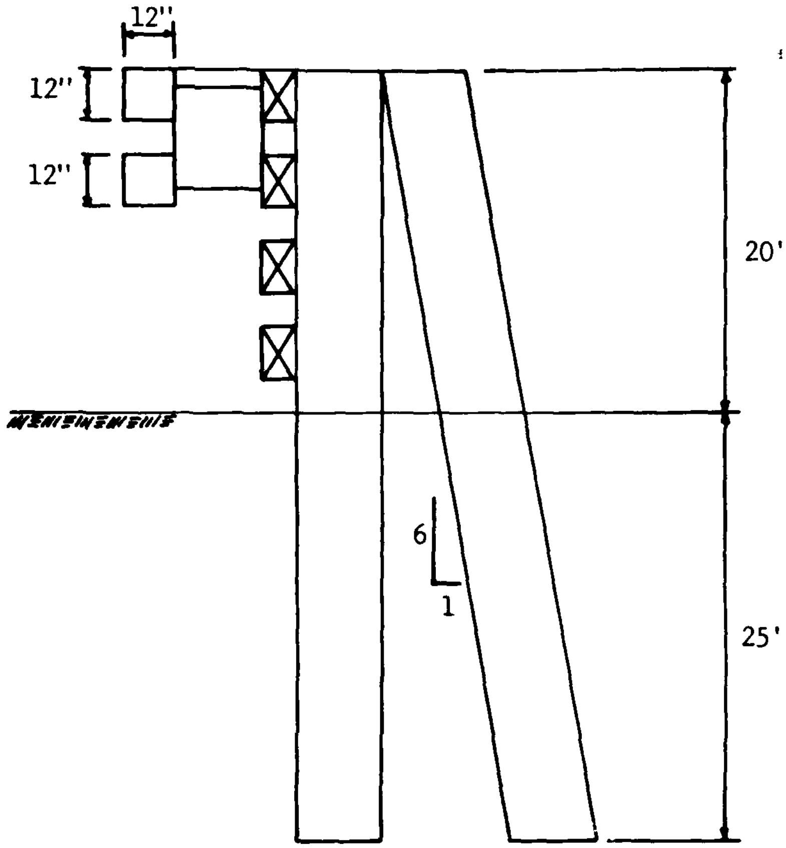
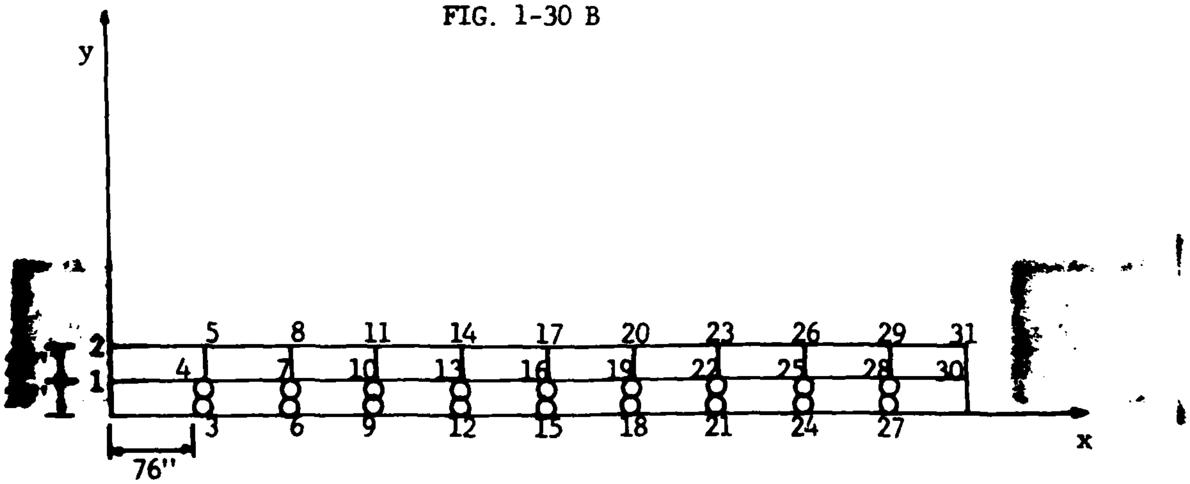


FIG. 1-30 B



CONNECTICUT RIVER BRIDGE

FIG. 1-31

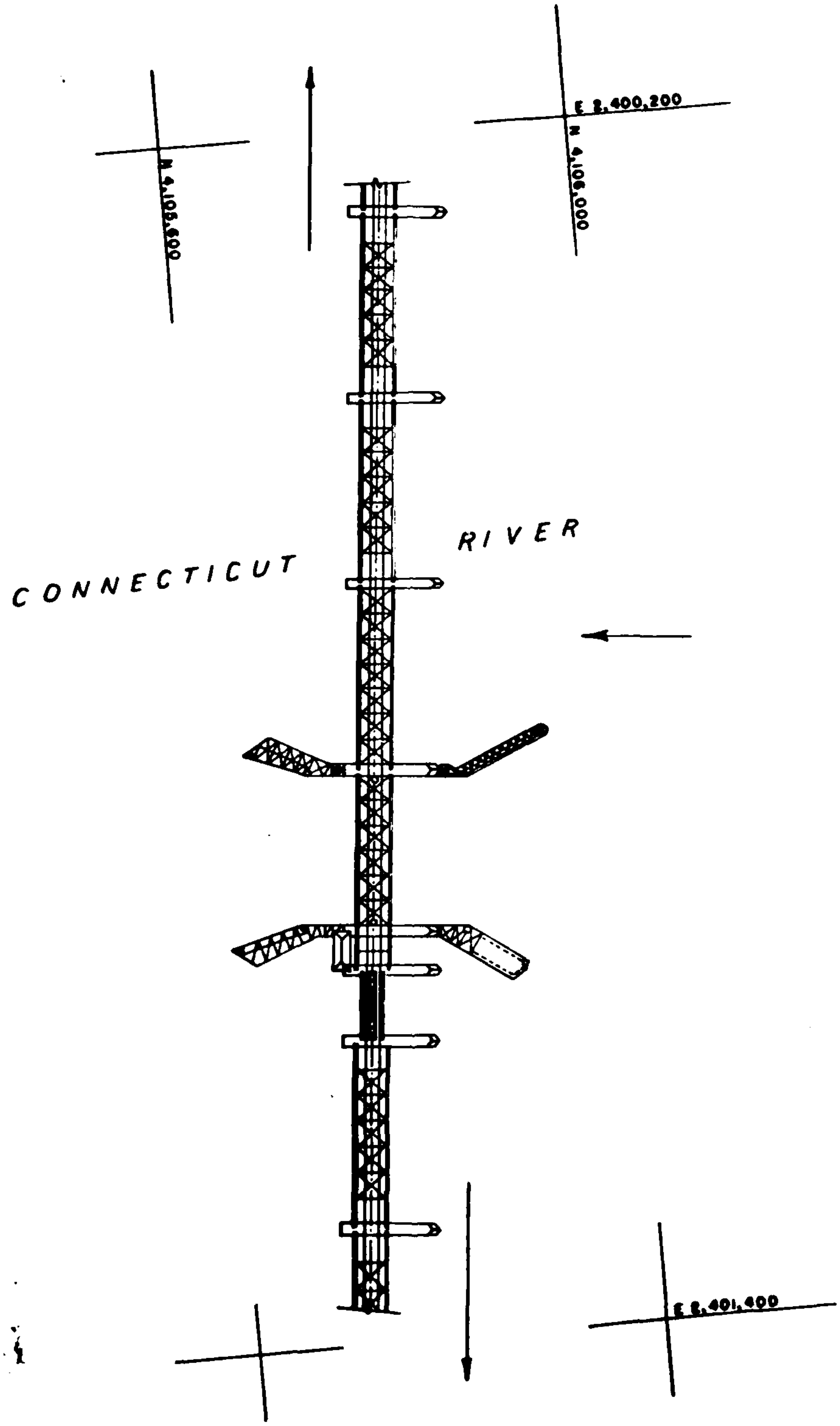
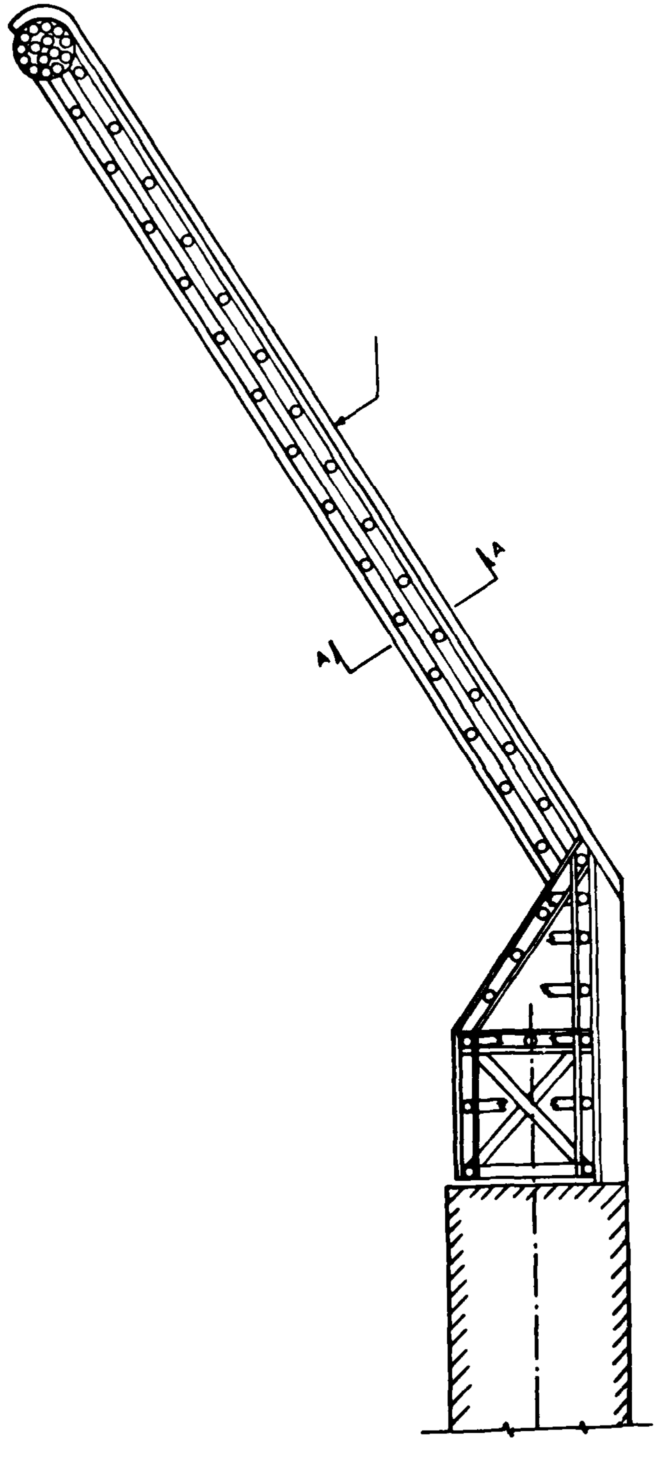
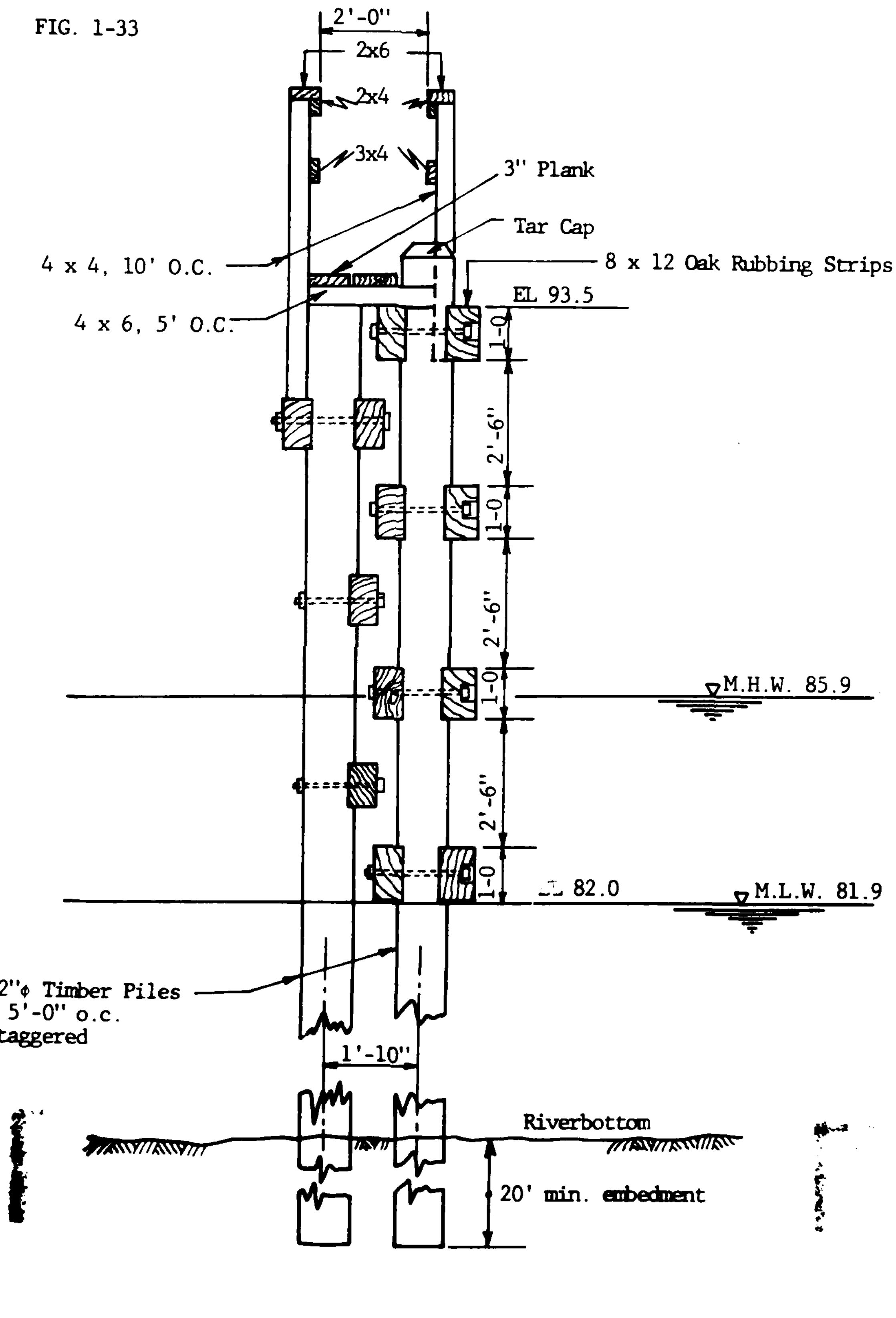


FIG. 1-32



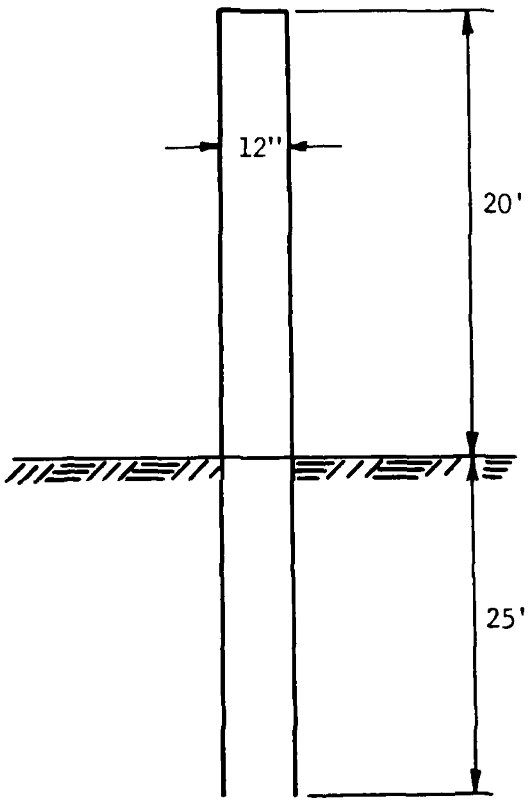
CON:CCTICUT RIVER BRIDGE

FIG. 1-33



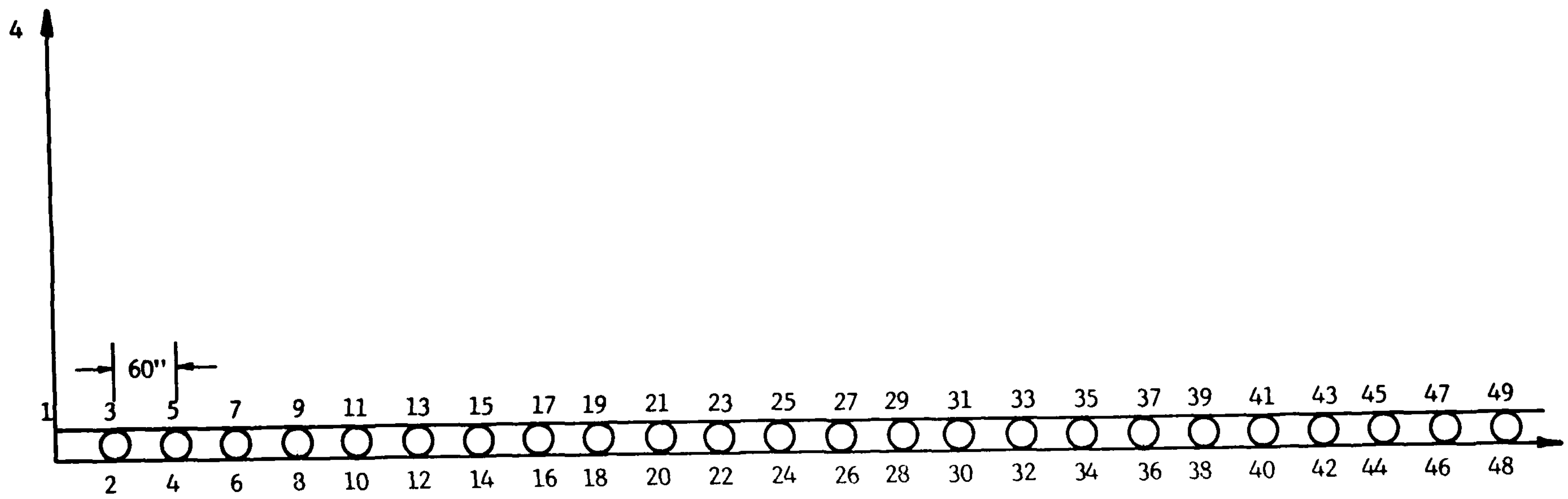
CONNECTICUT RIVER BRIDGE

FIG. 1-34



CONNECTICUT RIVER BRIDGE

FIG. 1-35



MARE ISLAND NAVY YARD

FIG. 1-36

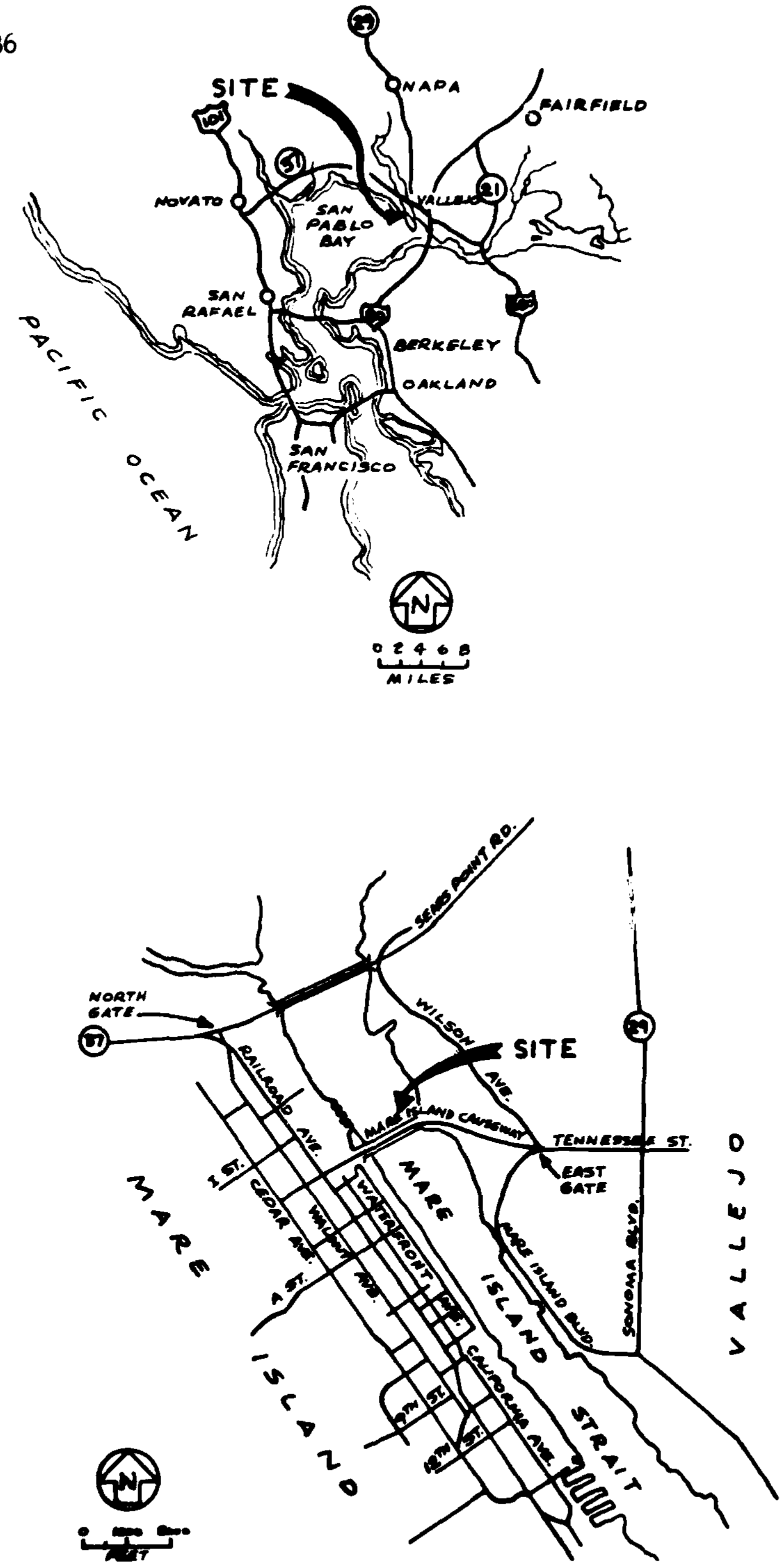
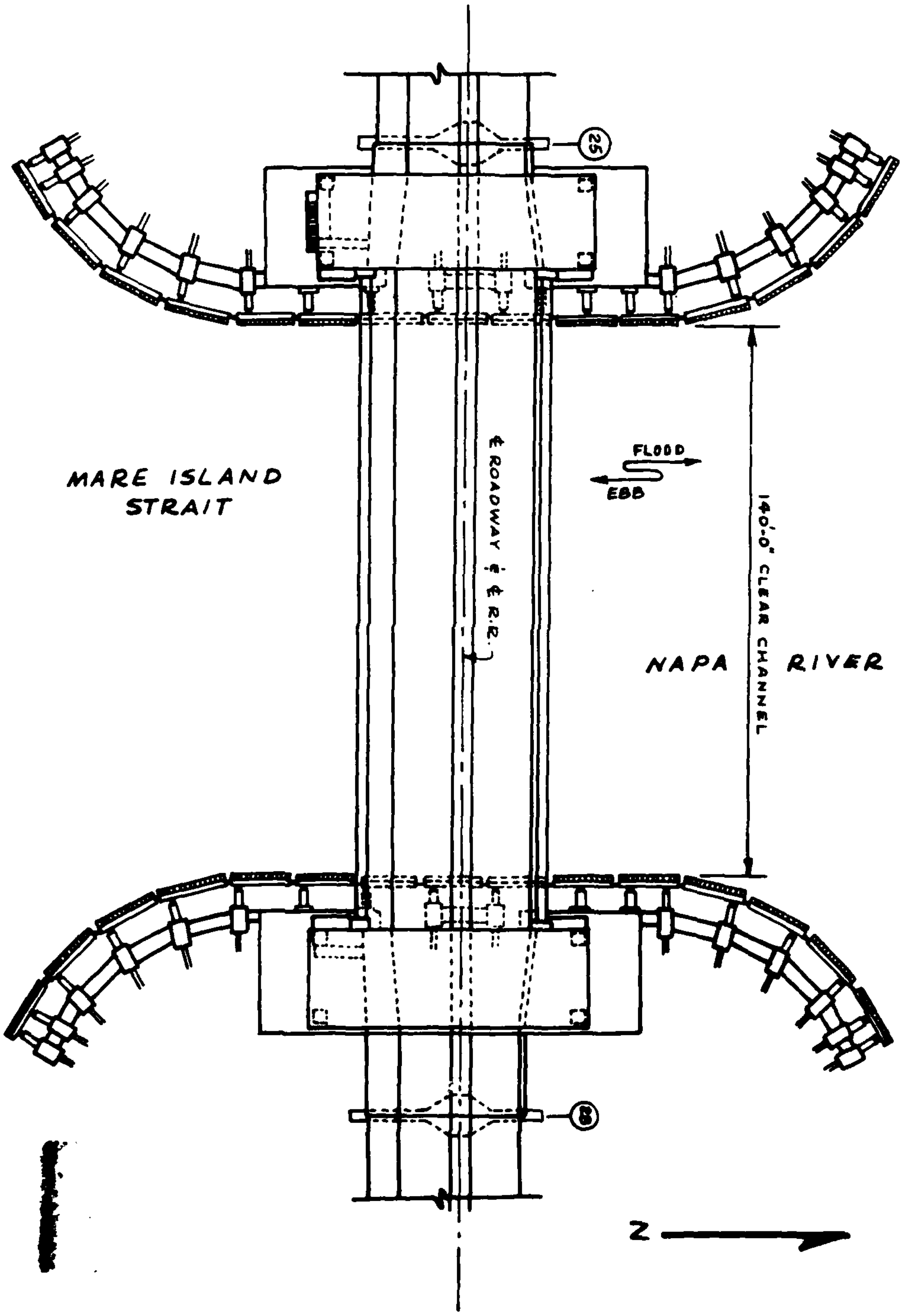
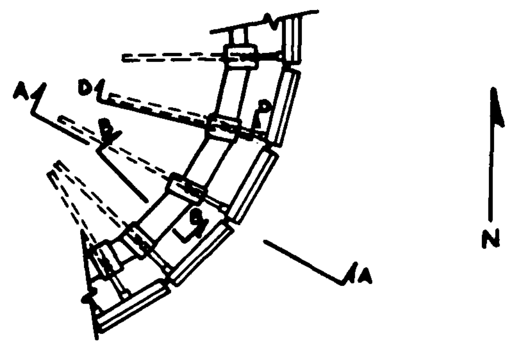

SITE
NAPA
FAIRFIELD
NOVATO
SAN PABLO BAY
VALLEJO
SAN RAFAEL
BERKELEY
OAKLAND
SAN FRANCISCO
PACIFIC OCEAN
0 2 4 6 8
MILES
SEARS POINT RD.
NORTH GATE
WILSON AVE.
SITE
RAILROAD AVE.
MARE ISLAND CAUSEWAY
TENNESSEE ST.
EAST GATE
I ST.
CEDAR AVE.
WALNUT AVE.
WATERFRONT AVE.
MARE ISLAND
MARE ISLAND STRAIT
MARE ISLAND BLVD.
SONOMA BLVD.
VALLEJO
A ST.
CALIFORNIA AVE.
8TH ST.
12TH ST.

FIG. 1-37

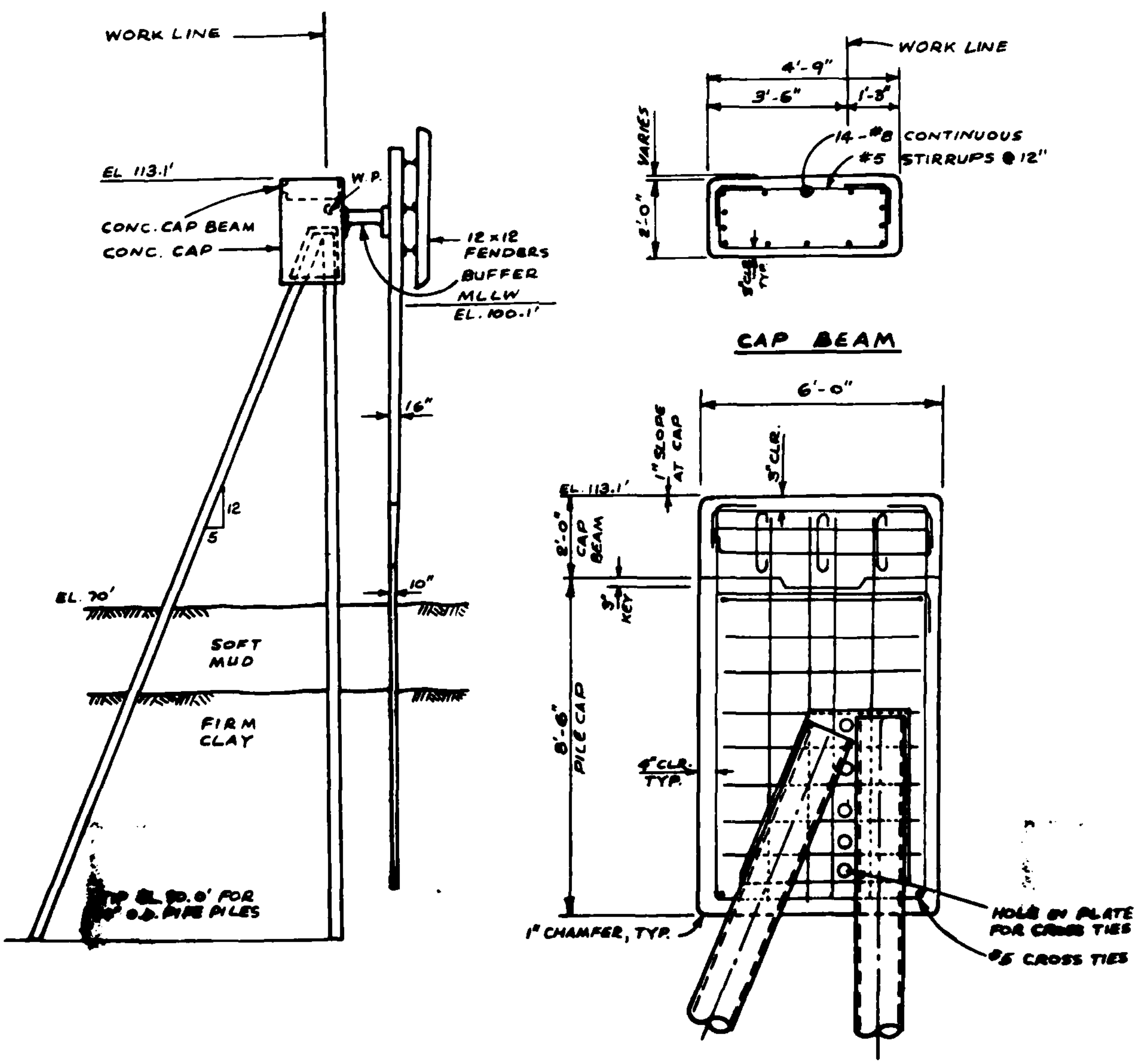


25
MARE ISLAND
STRAIT
℄ ROADWAY & ℄ R.R.
FLOOD
EBB
140'-0" CLEAR CHANNEL
NAPA RIVER
N

FIG. 1-38



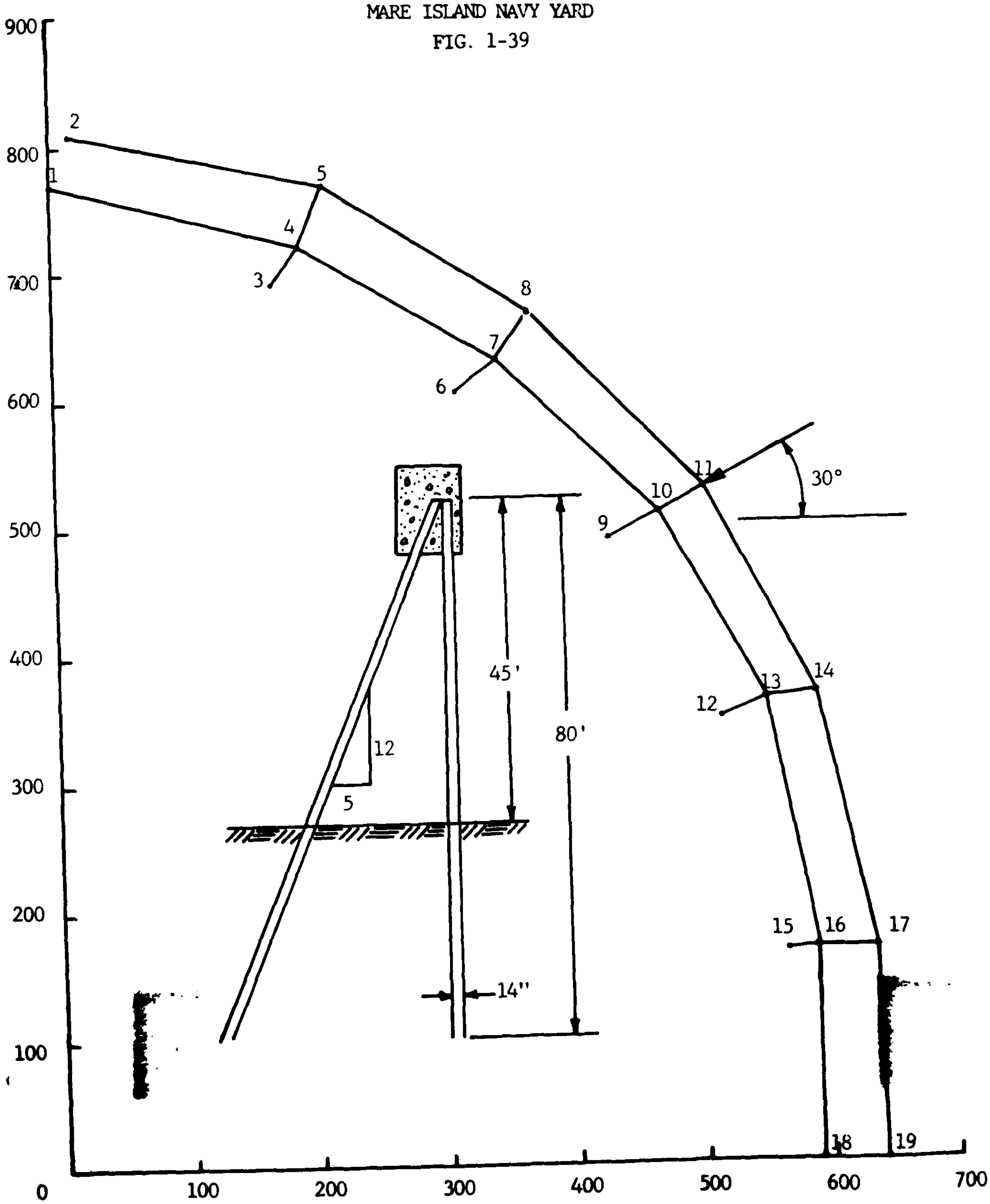
A
D
B
A
N



WORK LINE
EL 113.1'
W.P.
CONC. CAP BEAM
CONC. CAP
12 x 12
FENDERS
BUFFER
MLLW
EL. 100.1'
16"
12
5
EL. 70'
10"
SOFT
MUD
FIRM
CLAY
O.D. PIPE PILES
WORK LINE
4'-9"
3'-6"
1'-3"
VARIES
14-#8 CONTINUOUS
#5 STIRRUPS @ 12"
2'-0"
3" CLR. TYP.
CAP BEAM
6'-0"
1" SLOPE AT CAP
3" CLR.
EL. 113.1'
2'-0" CAP BEAM
3" KEY
8'-6" PILE CAP
4" CLR. TYP.
1" CHAMFER, TYP.
HOLE IN PLATE FOR CROSS TIES
#5 CROSS TIES

MARE ISLAND NAVY YARD

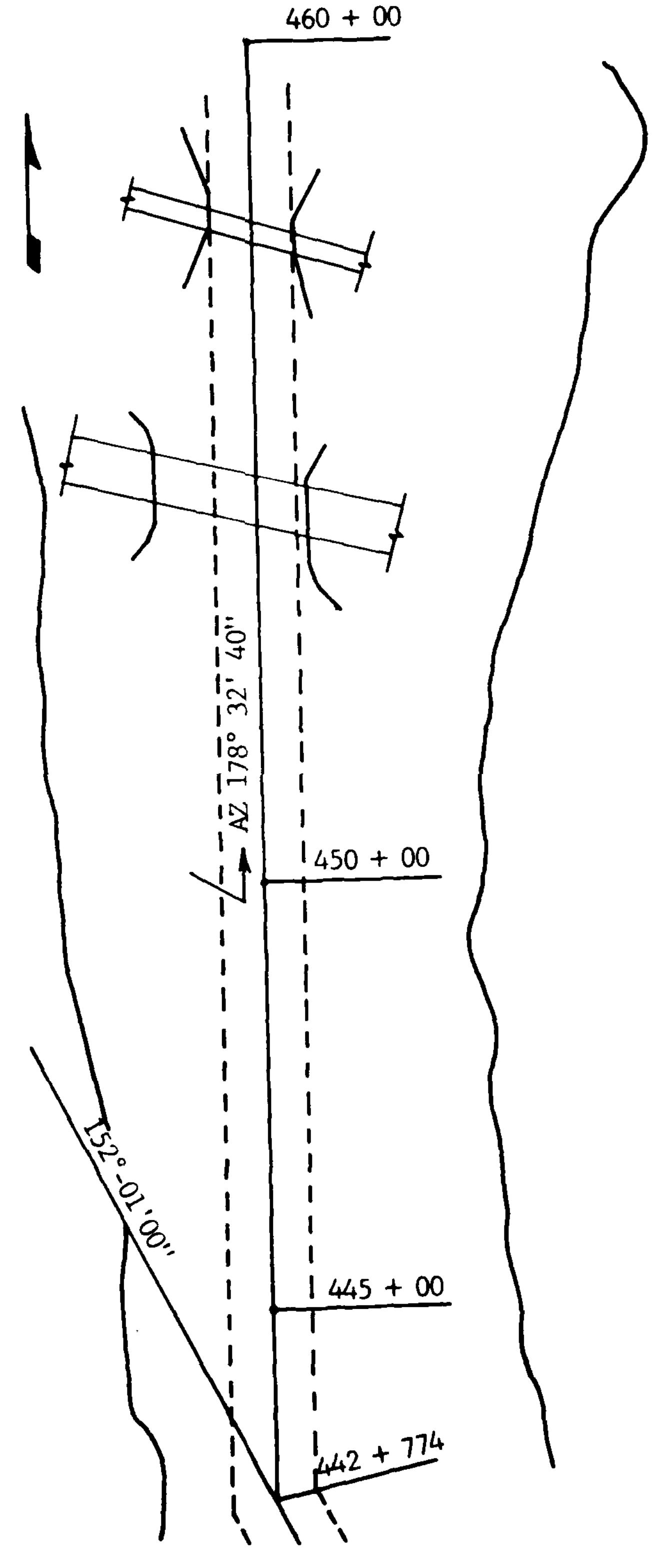
FIG. 1-39



DUWOMISH CHANNEL BRIDGE

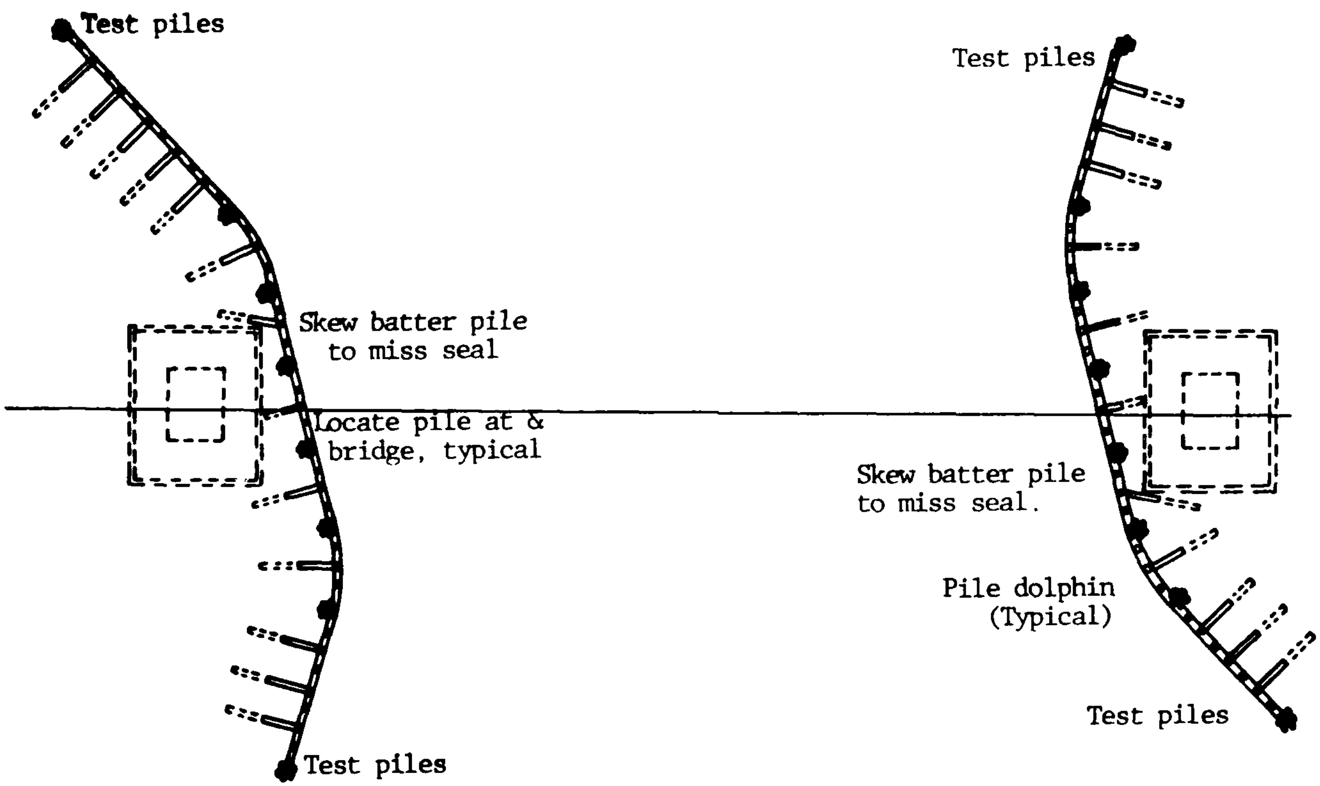
DUMOWISH CHANNEL BRIDGE

FIG. 1-40



DUMOMISH CHANNEL BRIDGE

FIG. 1-41



AD-A106 771 CIVIL DESIGN INC MORRISTOWN NJ F/G 13/13
LABORATORY MODEL TESTING OF BRIDGE PROTECTIVE SYSTEMS AND DEVIC--ETC(U)
SEP 81 K N DERUCHER DOT-CG-908665-A
UNCLASSIFIED 523-01-CG USCG-N-1-81 NL

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12 81
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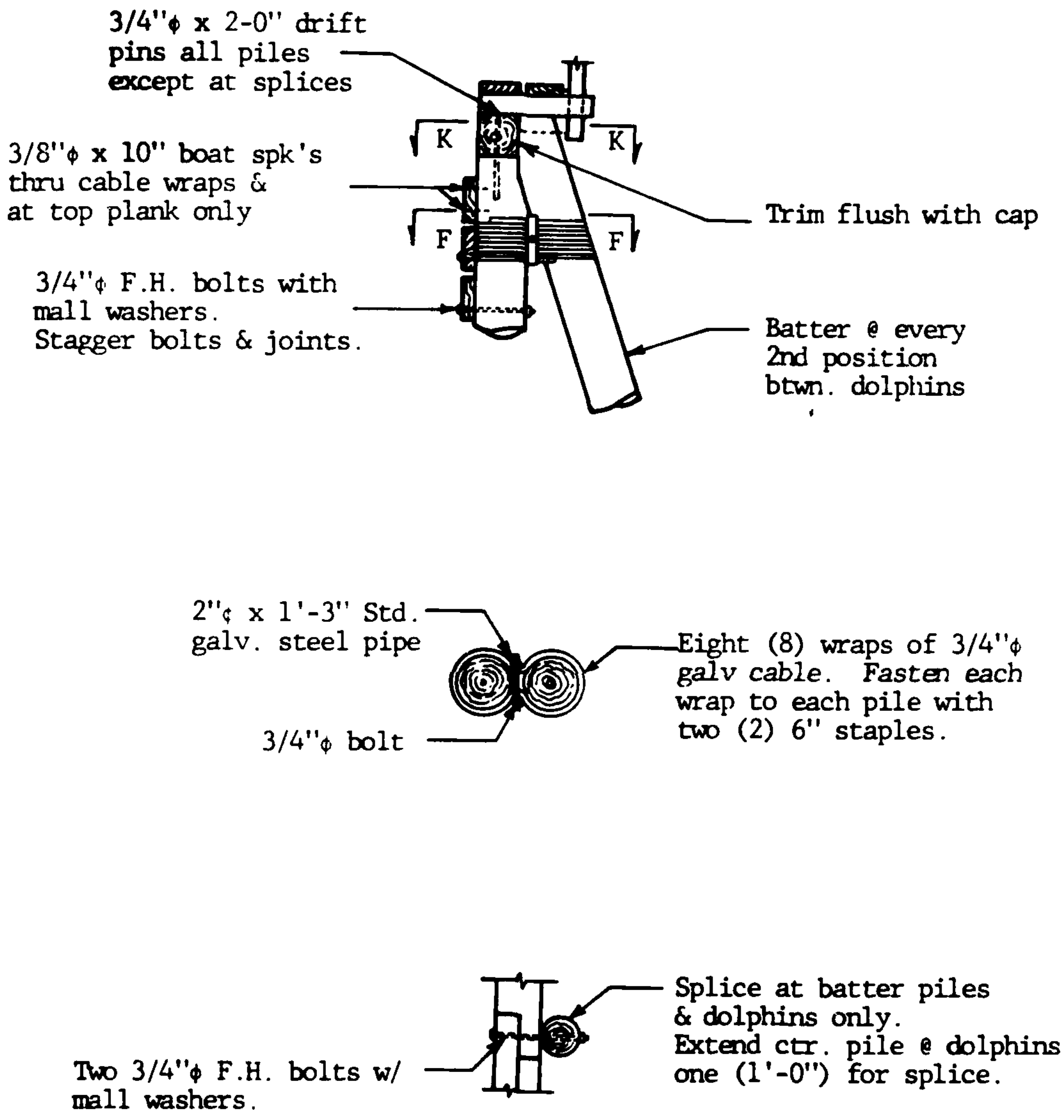
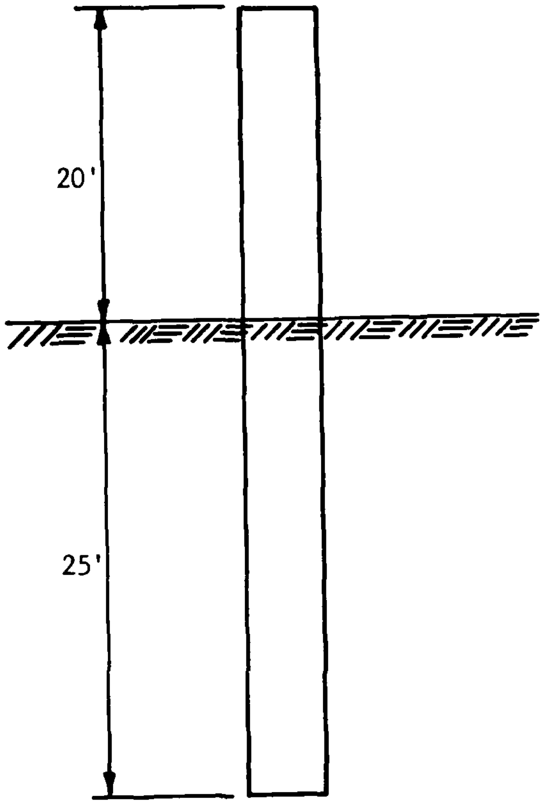
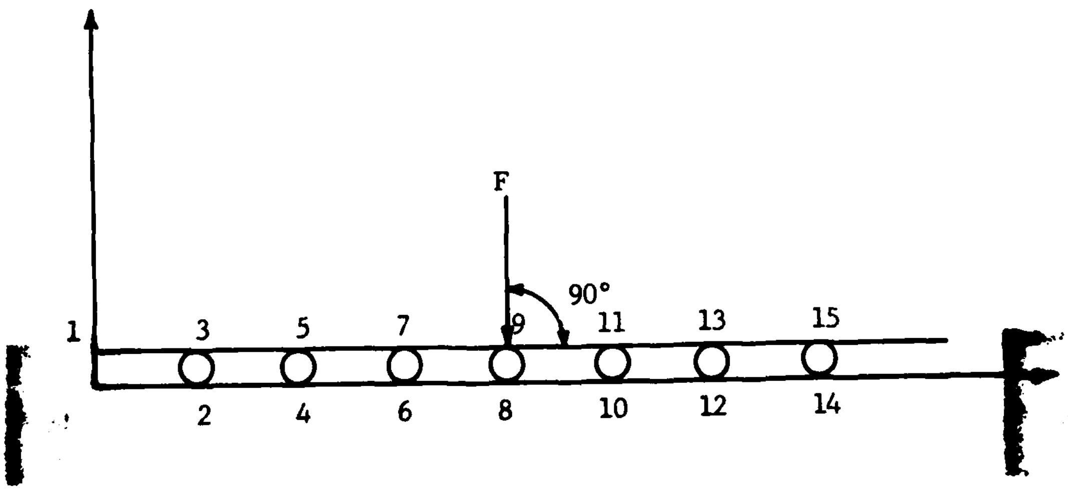
DUWOMISH CHANNEL BRIDGE

FIG. 1-42



DUWOMISH CHANNEL BRIDGE
FIG. 1-43





APPENDIX B

NAPA RIVER BRIDGE - MODELING ASSUMPTIONS
TABLE 1-1

A. GENERAL MATERIAL DATA

1. Subgrade modulus - 172.8 KSF (low end of organic skty clay)
2. Torsional modulus - 75 ksi (Wood)
3. Modulus of elasticity - 1200 ksi (Wood)

B. PILE DATA (2 VERTICAL PILE; 1 BATTERED)

1. Slope - .2500
2. Length - vertical 50.0'; battered 51.5'
3. Cantilever length - vertical - 20'; battered 20.6'
4. Circular pile X & Y projection - 20.88"
5. X & Y moment of inertia - 9330"^{4}
6. Polar moment - 18660"^{4}
7. Cross-sectional area - 342"^{2}
8. Yield stress - 5 ksi
9. Neutral - axis - outer fiber length - 10.44"
10. Vertical load = 0

C. SYSTEM DATA

1. No. pile groups - 9
2. No. of fenders - 9
   Loed SF600 a = 0.02 b = -0.39 c = 5.7
3. Spacing between groups - 90"
4. Support beam mod of elas. - 1200 ksi (Wood)
5. Support beam moment of I - 768"^{4}
6. Support beam area - 144"^{2}
7. Fender beam mod of elast. - 1200 ksi (Wood)
8. Fender beam moment of I - 3456"^{4}
9. Fender beam area - 288"^{2}

THIRD STREET BRIDGE - WITHOUT FENDERS
TABLE 1-2

MODELING ASSUMPTIONS:

A. GENERAL DATA

1. Subgrade modulus - 172.8 KSF (100psi)
2. Modulus of rigidity - 75 ksi Timber
3. Modulus of elasticity - 1,200 ksi Timber

B. PILE DATA (3 PER GROUP)

1. Total length - 45'
2. Cantilever length - 20'
3. Circular pile X & Y projection - 16'
4. X & Y mom. of I - $3217''^4$
5. Polar mom. - $6434.''^4$
6. Cross-sectional area - $201''^2$
7. Yield stress timber - 5 ksi
8. Neutral axis to outer fiber length - 8"
9. Vertical load - 0

C. SYSTEM DATA

1. Pile groups - 13 (all vertical)
2. Fender - none
3. Spacing between pile groups - 90"
4. Support beam modulus of elas. - 1200 ksi
5. Support beam mom. of I - $1728''^4$
6. Support beam area - $144''^2$

D. GENERAL PILE DATA

1. Elevated platform (3)
2. Fixed pile cap (1)
3. End bearing (2)
4. All vertical piles (1)
5. No sample points (0)

THIRD STREET BRIDGE - MODELING ASSUMPTIONS - WITHOUT FENDERS
TABLE 1-3

A. MATERIAL PROPERTIES

1. Subgrade modulus - 172.8 KSF
2. Modulus of rigidity - 75 ksi (Timber piles)
3. Modulus of elasticity - 1,200 ksi (Timber piles)

B. PILE DATA (3 PER GROUP)

1. Total length - 45'
2. Cantilever length - 20'
3. Circular pile X & Y projection - 16"
4. X & Y moment of inertia - $3217''^4$
5. Polar moment - $6434''^4$
6. Cross-sectional area - $201''^2$
7. Yield stress - 5 ksi (Timber)
8. Neutral axis to outer fiber length - 8"
9. Vertical load - 0

C. SYSTEM DATA

1. Pile groups - 13 (all vertical)
2. Fender - Lord 5F-600
   a = 0.02 b = -0.39 c = 5.7
3. Spacing between pile groups - 90"
4. Support beam mod. of elasticity - 1200 ksi (Timber)
5. Support beam moment of I - $6912''^4$ (4-2x12 wales)
6. Support beam area - $576''^2$ (4-12x12 wales)
7. Fender beam mod. of elasticity - 1200 ksi (Timber)
8. Fender beam moment of I - 3456 $''^4$ (2-12x12 robbing st.)
9. Fender beam area - $288''^2$

STEAMBOAT RIVER BRIDGE - MODELING OUTLINE
TABLE 1-4

A. MATERIAL PROPERTIES
   1. Subgrade modulus - 175.0 k/ft^3
   2. Modulus of rigidity - 75 ksi (Timber piles)
   3. Modulus of Elasticity - 1200 ksi (Timber piles)

B. PILE DATA (2 vertical separated each other)
   1. Total length - 50'
   2. Cantilever length - 25'
   3. Spacing between piles - 36"
   4. Circular pile X & Y projection - 12"
   5. X & Y moments of inertia - $10.8''^4$
   6. Polar moment inertia - $2036''^4$
   7. Cross-sectional area - $113''^2$
   8. Yield stress - 5 ksi (Timber)
   9. Neutral axis to outer fiber length - 6"
   10. Vertical load - 0.0 k

C. SYSTEM DATA
   1. No. of pile groups - 11
   2. No. of fenders
   3. Spacing between pile groups - 48"
   4. Support beam modulus of elasticity - 1200 ksi (Timb r)
   5. Support beam moment of inertia - 15 @ 4 x 12 = 960 in^4
   6. Support beam area - 15 @ 4 x 12 = 720 in^2

HOQUIAM RIVER BRIDGE - MODELING ASSUMPTIONS
TABLE 1-5

A. MATERIAL PROPERTIES

1. Subgrade modulus - 172.8 KSF
2. Modulus of rigidity - 75 ksi (Timber piles)
3. Modulus of elasticity - 1200 ksi (Timber piles)

B. PILE DATA (1 VERTICAL, 1 BATTERED)

1. Total length - 45'
2. Cantilever length - 20'
3. Circular pile X & Y projection - 12"
4. X & Y moments of inertia - $1018''^4$
5. Polar mom. - $2036''^6$
6. Cross-sectional area - $113''^2$
7. Yield stress - 5 ksi (Timber)
8. Neutral axis to outer fiber length - 6"
9. Vertical load - 0.0

C. SYSTEM DATA

1. Pile groups - 21
2. Fender - Lord 5F-600 (a = 0.02, b = -0.39, c = 5.7)
3. Spacing between pile groups - 60"
4. Support beam modulus of elas. - 1200 ksi (Timber)
5. Support beam mom. of I - $1 @ 12 \times 12 = 1728''^4$

$$\begin{aligned} 1 @ 10 \times 12 &= 1000''^4 \\ 5 @ 4 \times 12 &= \underline{320''^4} \\ & 3048''^4 \end{aligned}$$

6. Support beam area - $312''^2$
7. Fender beam mod. of elasticity - 1200 ksi (Timber)
8. Fender beam mom. of I - $3456''^4$ (2 - 12 x 12 Rubb. St.)
9. Fender beam area - $288''^2$

SCHUYKILL RIVER BRIDGE - MODELING OUTLINE
TABLE 1-6

A. MATERIAL PROPERTIES

1. Subgrade modulus - 172.8 KSF
2. Modulus of rigidity - 75 ksi (Timber piles)
3. Modulus of elasticity - 1200 ksi (Timber piles)

B. PILE DATA (5 PILES CONVERTED TO A 2 PILE BATTERED CONFIGURATION.)

1. Total length - 45'
2. Cantileber length - 20'
    a. Battered pile
        1. Slope - .1667
        2. Total length - 45.6'
        3. Cantilever length - 20.3'
3. Circular pile X & Y protection - 14"
4. X & Y moment of I - $1886''^4$
    a. vert. - 3 @ 1886 = $5658''^4$
    b. bat. - 2 @ 1886 = 3772.
5. Polar mom
    a. vert - $11316''^4$
    b. bat - $7544''^4$
6. Cross sectional area - 154
    a. vert - 3 @ 154 = $462''^2$
    b. bat - 2 @ 154 = $308''^2$
7. Yield stress - 5 ksi (Timber)
8. Neutral axis to outer fiber length - 7"
9. Vertical load = 0.0

CONNECTICUT RIVER BRIDGE (ALTERNATIVE A) - MODELING ASSUMPTIONS
TABLE 1-7

A. MATERIAL PROPERTIES

1. Subgrade modulus - 172.8 KSF
2. Modulus of rigidity - 75 ksi (Timber piles)
3. Modulus of elasticity - 1,200 ksi (Timber piles)

B. PILE DATA

1. Total length - 45'
2. Cantilever length - 20' (above mod line)
3. Circular pile X & Y projection - 12"
4. X & Y moments of inertia - $1018''^4$
5. Polar moment - $2036''^4$
6. Cross-sectional area - $113''^2$
7. Neutral axis to outer fiber - 6"
8. Vertical load - 0

C. SYSTEM DATA

1. Pile groups - 24 (all vertical piles)
2. Fender - Lord 5F-600
   $a = 0.02$ $b = -0.39$ $c = 5.7$
3. Spacing between pile groups - 60"
4. Support beam mod. of elasticity - 1200 ksi (Timber)
5. Support beam moment of I - $512''^4$/ Oak rubbing strip (x8)
6. Support beam area - 96" (x8)
7. Fender beam mod. of elasticity - 1200 ksi (Timber)
8. Fender beam moment of I - $3456''^4$ (2-12x12 Rubbing strips)
9. Fender beam area - $288''^2$

MARE ISLAND NAVY YARD - MODELING ASSUMPTIONS
TABLE 1-8

A. GENERAL MATERIAL DATA

1. Subgrade modulus - 172.8 KSF (low end of organic silty clay)
2. Torsional modulus - 11,500 ksi (Steel pipe)
3. Modulus of elasticity - 30,000 ksi (Steel pipe)

B. PILE DATA (1 VERTICAL PILE; 1 BATTERED)

1. Slope - .4167
2. Length - vertical 80.1'; battered 48.86'
3. Cantilever length - vertical 45.1; battered 48.86'
4. Circular pipe X & Y projection - 14"
5. X & Y Moment of inertia - 1886.0"^{4} (14"O.D. x 1/2") Filled
6. Polar moment - 3772.0"^{4} with
7. Cross-sectional area - 154.0"^{2} sand
8. Yield stress - 36 ksi
9. Meutral - axis - outer fiber length - 7"
10. Vertical load - 55 kips

C. SYSTEM DATA

1. No. pile groups - 5
2. No. fender - 5

   Lord 5 F600 a = 0.02 b = -0.39 c = 5.7
3. Spacing between groups - 172"
4. Support beam mod. of elast. - 4,000 ksi
5. Support beam moment of I - 99,999"^{4}
6. Support beam area - 10,000"^{2}
7. Fender beam mod. of elast. - 30,000 ksi
8. Fender beam moment of I - 99,999"^{4}
9. Fender beam area - 10,000"^{2}

DUWAMISH CHANNEL BRIDGE - WITH FENDERS
TABLE 1-9

MODELING ASSUMPTIONS

A. MATERIAL PROPERTIES

1. Subgrade modulus - 172.8 KSF
2. Modulus of rigidity - 75 ksi (Timber piles)
3. Modulus of elasticity - 1200 ksi (Timber piles)

B. PILE DATA (1/Group Alt. A; 2/Group Alt. B)

1. Total Length - 50'
2. Cantilever length - 25'
3. Circular pile X & Y projection - 12"
4. X & Y Moments of inertia - $1018''^4$
5. Polar mom. - $2036''^4$
6. Cross-sectional area - $113''^2$
7. Yield Stress - 5 ksi (Timber)
8. Neutral axis to outer fiber length - 6"
9. Vertical load - 0.0

C. SYSTEM DATA

1. Pile groups - 7
2. Fender - Lord 5F-600
   a = 0.02 b = -0.39 c = 5.7
3. Spacing between pile groups - 60"
4. Support beam modulus of elas. - 1200 ksi (Timber)
5. Support beam mom. of I - 11 at (12x4) = $704''^4$
6. Support beam area - 11 at (12x4) = $528''^2$
7. Fender beam mod. of elasticity - 1200 ksi (Timber)
8. Fender beam moment of I - $3456''^4$ (2-12x2 robbing st.)

DUWAMISH CHANNEL BRIDGE - WITHOUT FENDERS
TABLE 1-10

MODELING ASSUMPTIONS:

A. GENERAL DATA

1. Subgrade modulus - 172.8 KSE (No soil data)
2. Molulus of rigidity - 75 ksi Timber
3. Modulus of elasticity - 1,200 ksi Timber

B. PILE DATA

1. Total length - 45'
2. Cantilever length - 20'
3. Circular pile X & Y projected length - 12"
4. X & Y mom. of I - 1018"^{4}
5. Polar mom. - 2036"^{4}
6. Cross-sectional area - 113"^{2}
7. Yield stress timber - 5 ksi
8. Neutral axis to outer fiber length - 6"
9. Vertical load - 0.0

C. SYSTEM DATA (alt. A)

1. Pile groups - 7 (all vertical)
2. Fender - none
3. Spacing between pile group - 60"
4. Support beam modulus of elas. - 1200 ksi
5. Support beam mom. of I - 288"^{4} (2x12)
6. Support beam area - 24"^{2}

D. SYSTEM DATA (alt. B)

1. Pile groups - 7 (1-vertical, 1-battered)
   a. Battered pile slope - .333
   b. Battered pile length - 47.4'
   c. Battered pile cantilever L. - 21.1'
2. Same support data

APPENDIX C

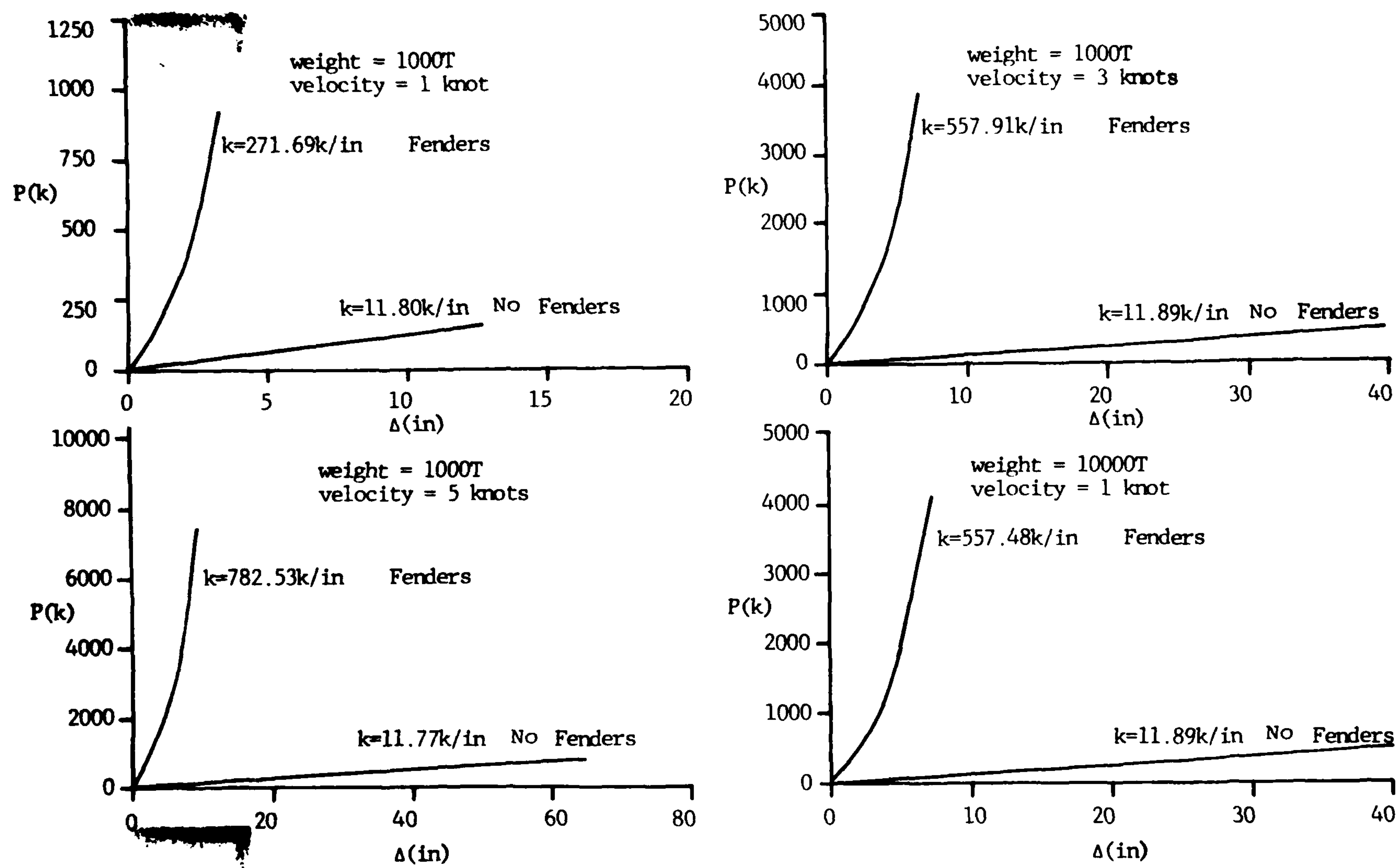


FIG. 1-44

Napa River Bridge

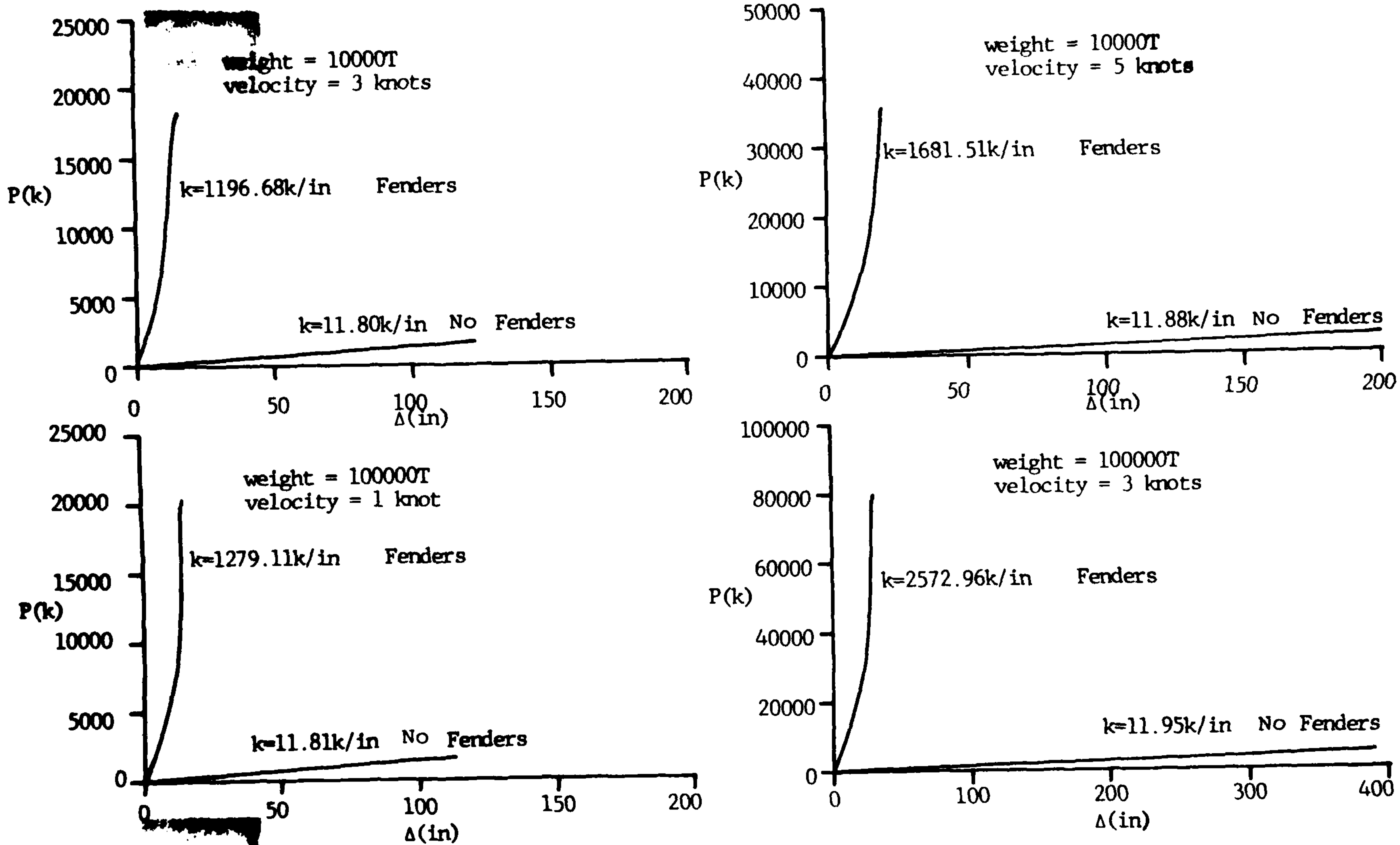


FIG. 1-45

Napa River Bridge

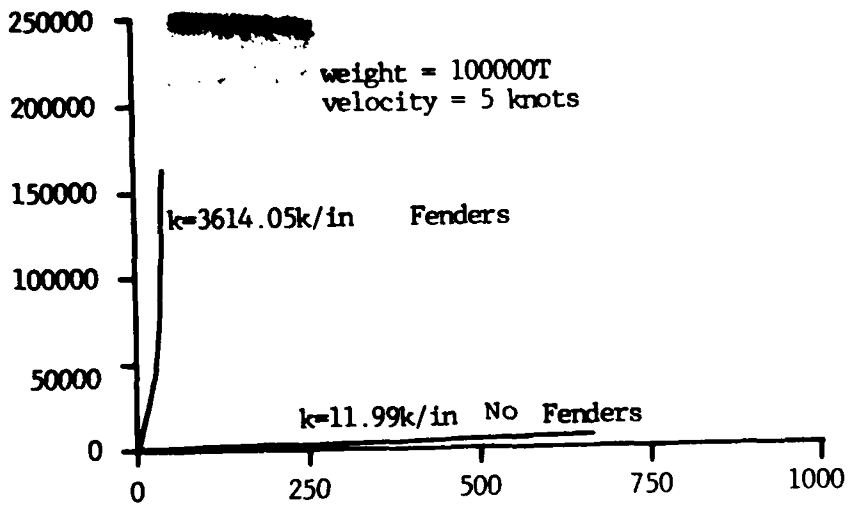


FIG. 1-46

Napa River Bridge

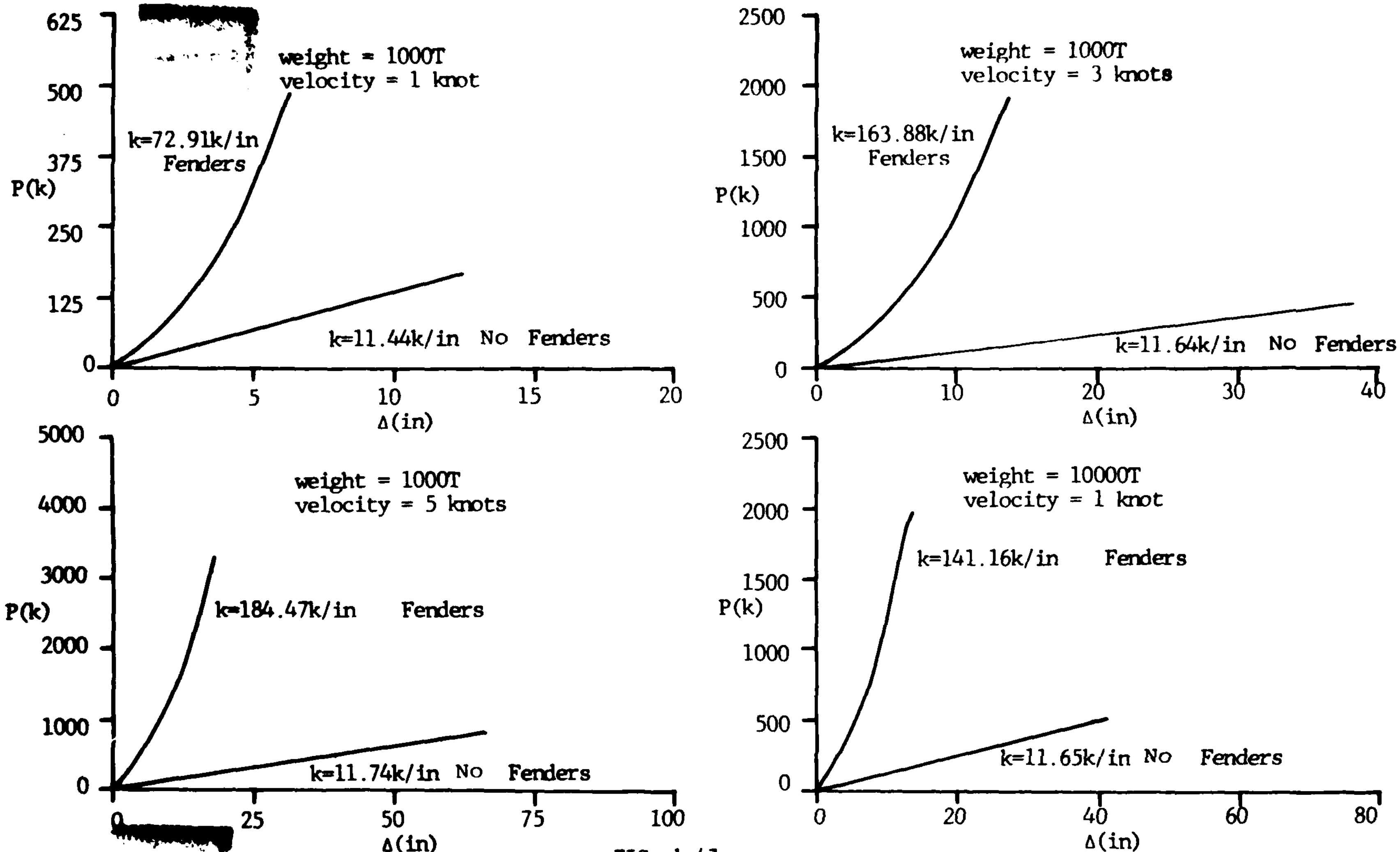


FIG. 1-47

Third Street Bridge

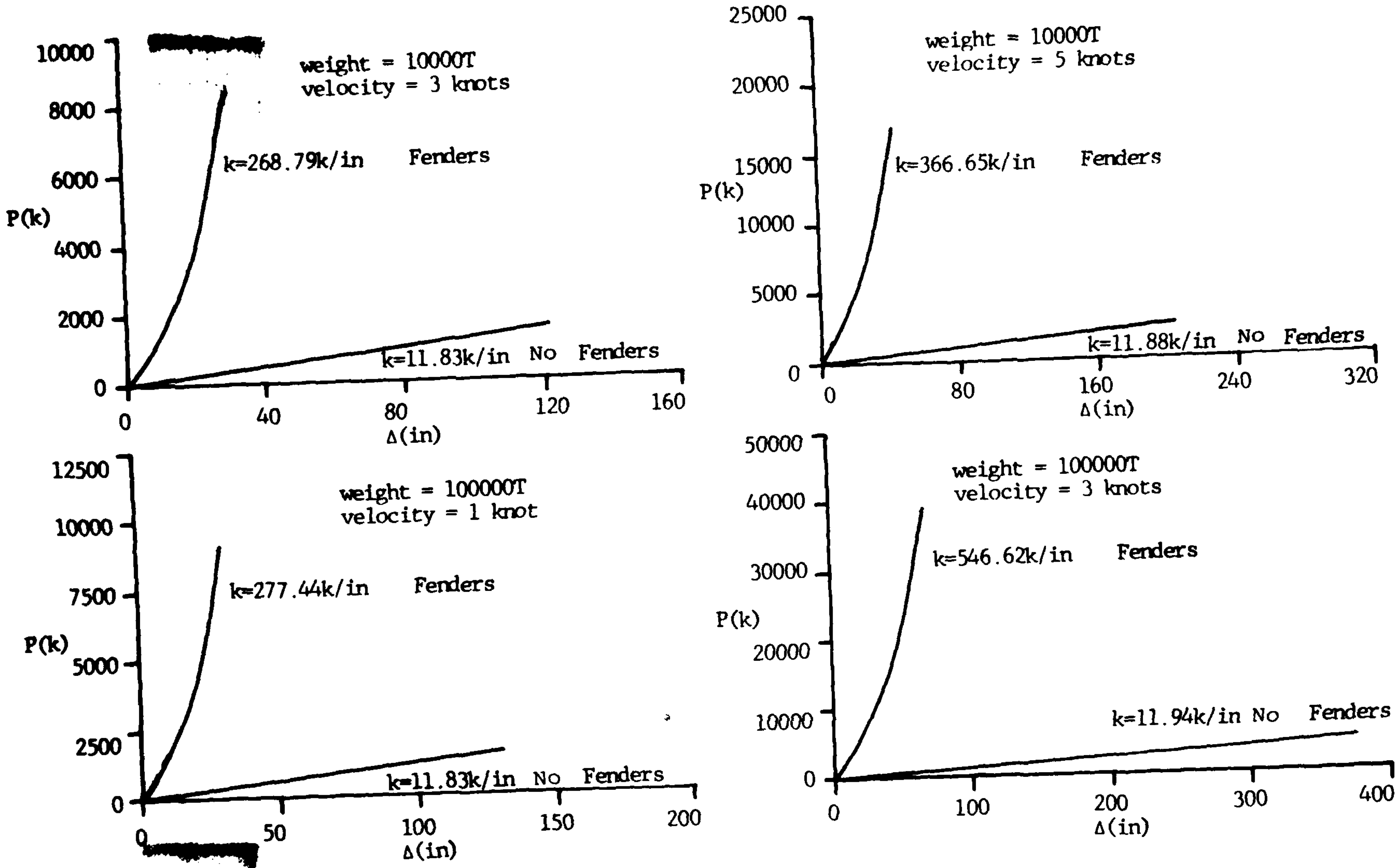


FIG. 1-48

Third Street Bridge

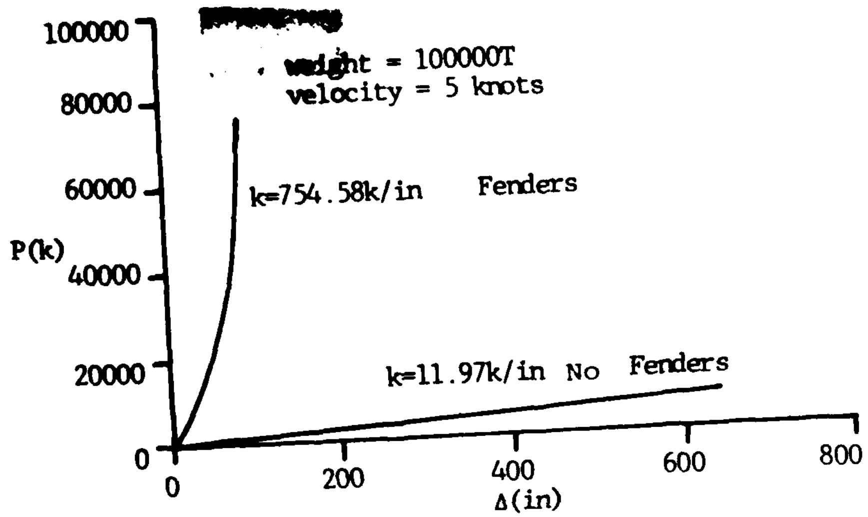


FIG. 1-49

Third Street Bridge

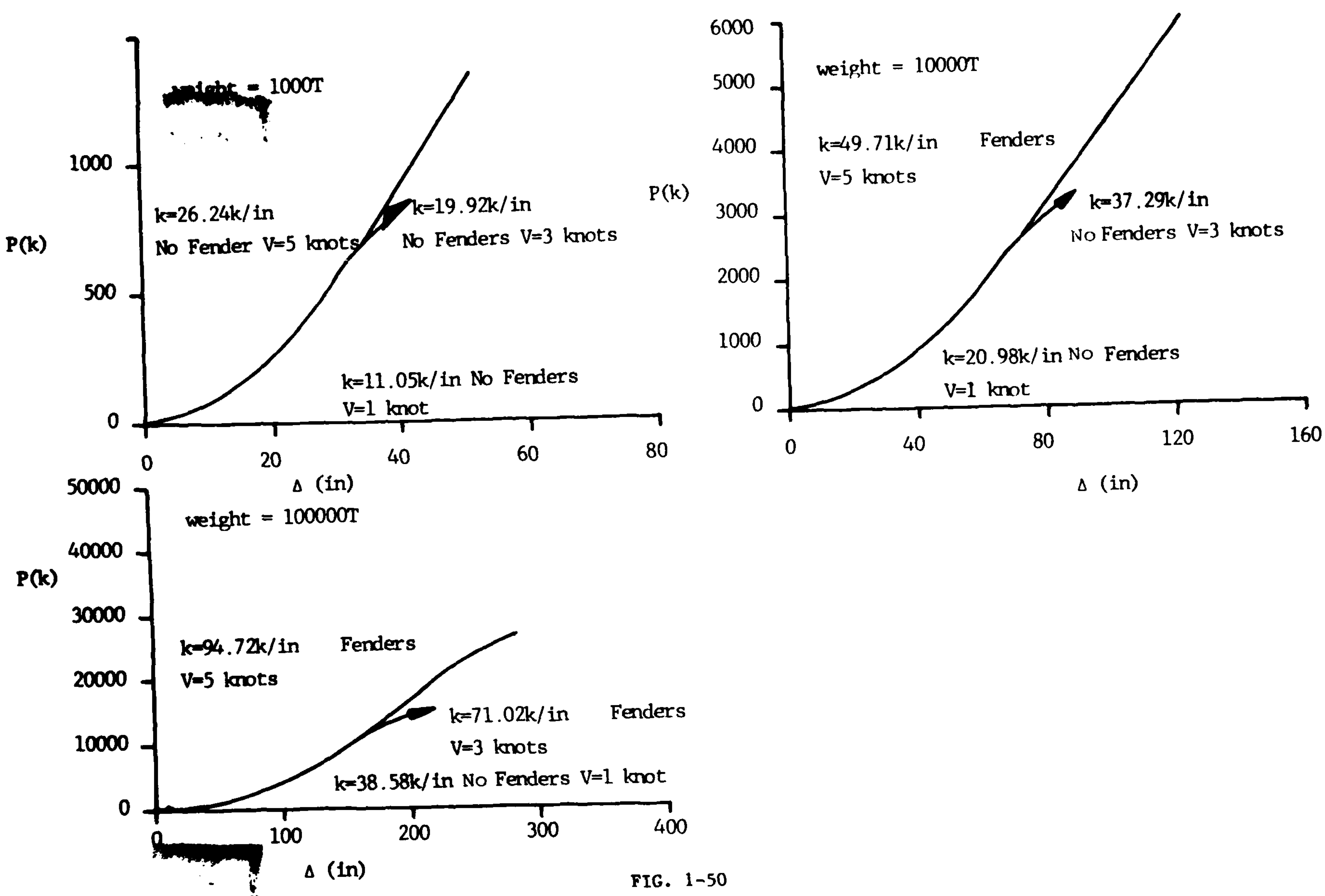


FIG. 1-50

Steamboat Slough Bridge

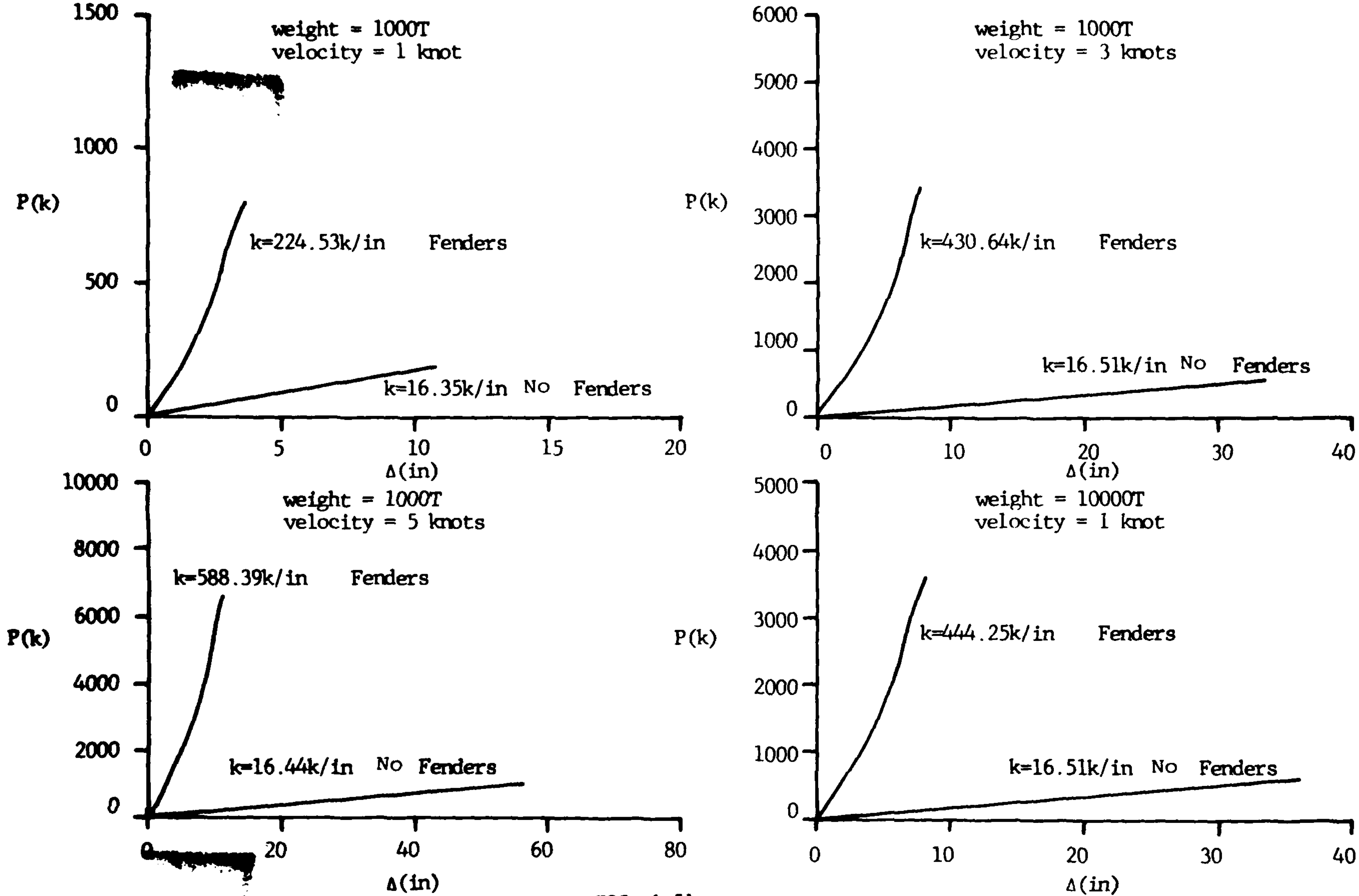


FIG. 1-51

Hoquiam River Bridge

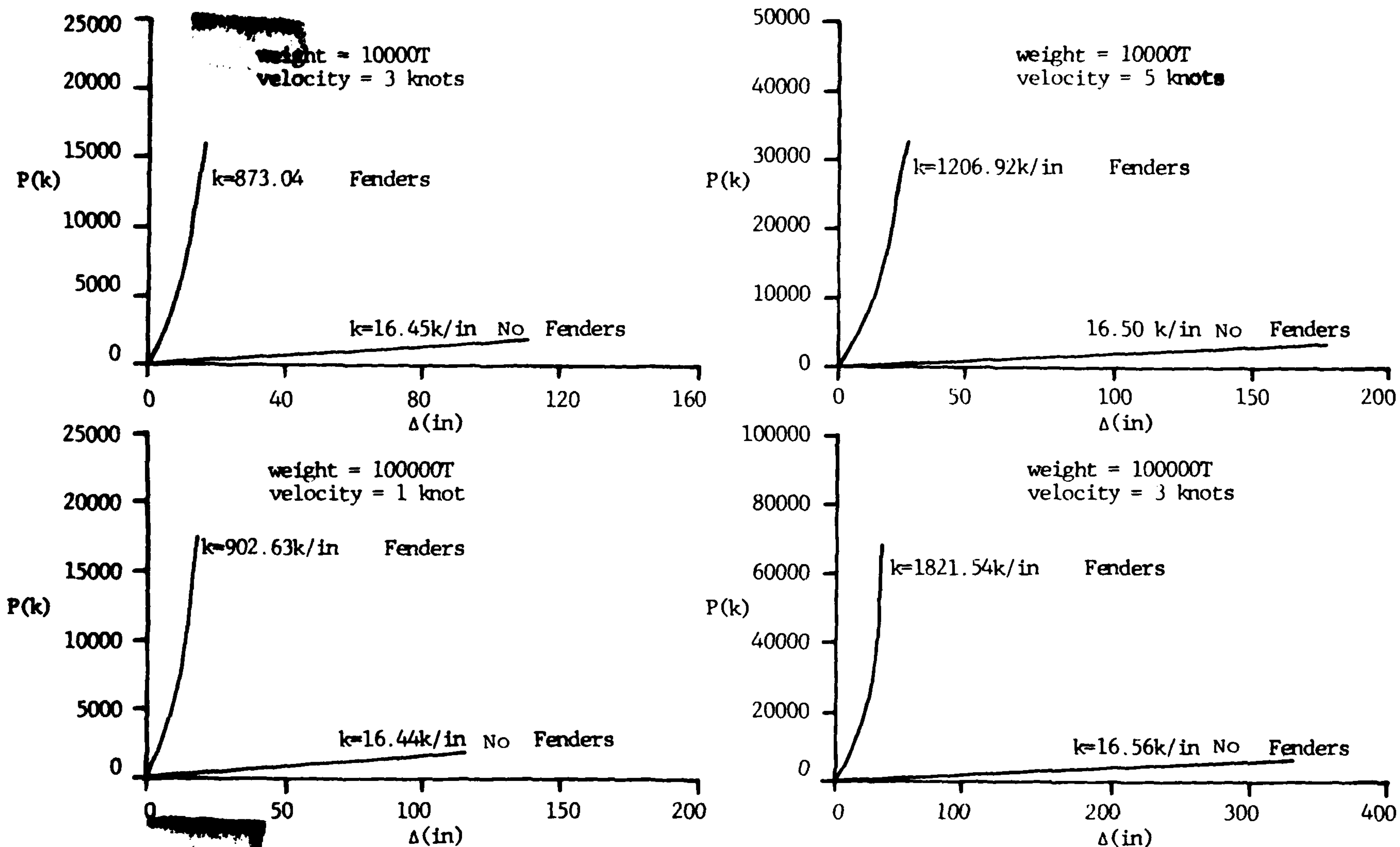


FIG. 1-52

Hoquiam River Bridge

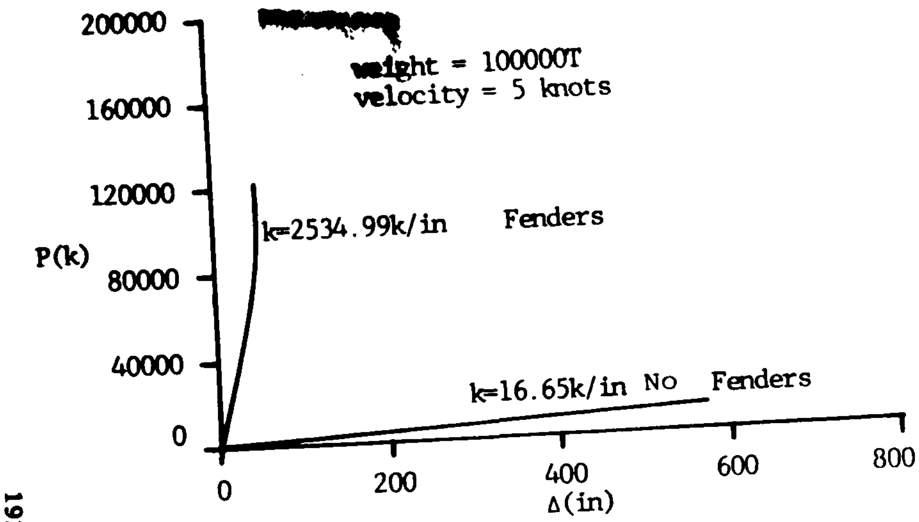


FIG. 1-53

Hoquiam River Bridge

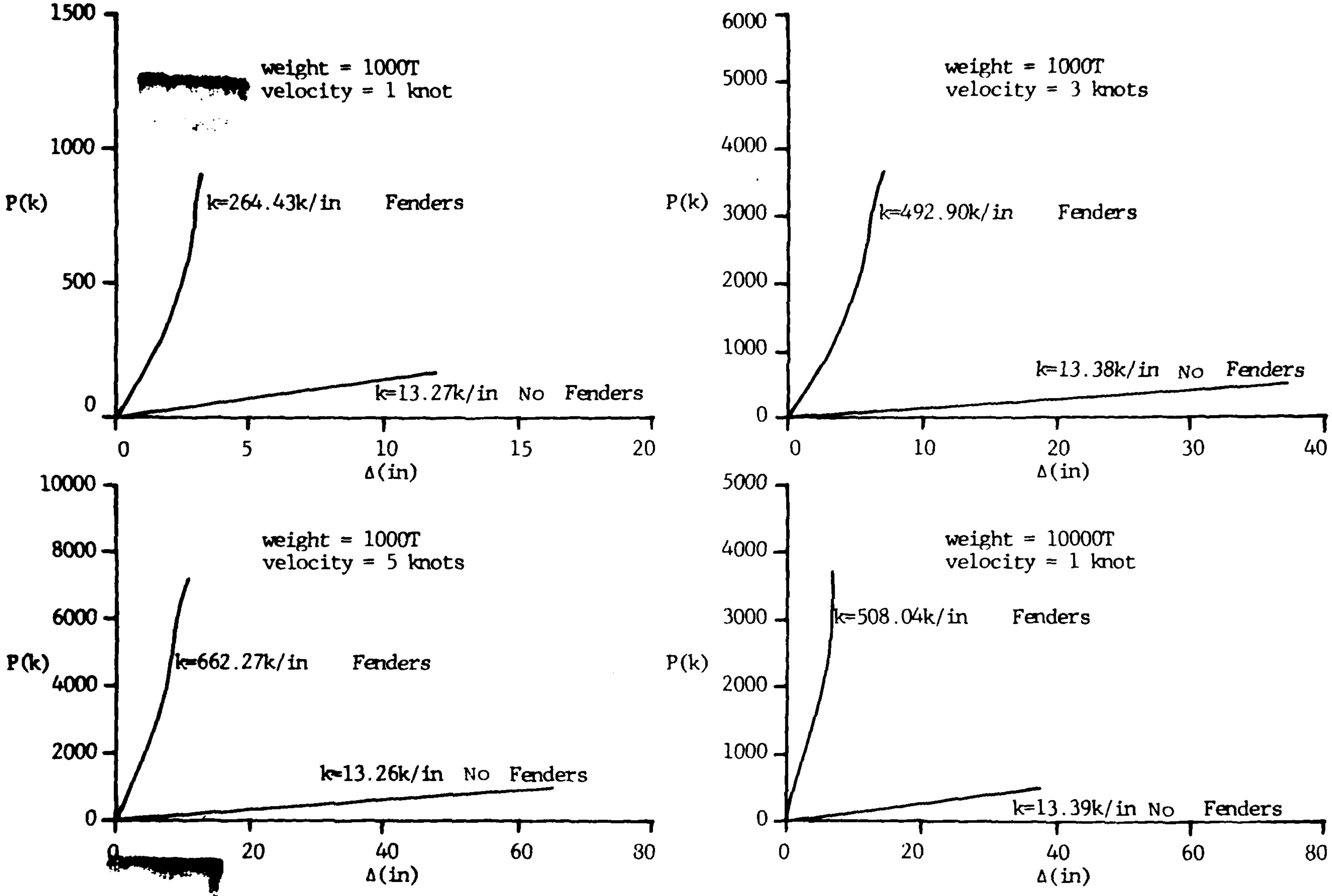


FIG. 1-54

Schuylkill River Bridge

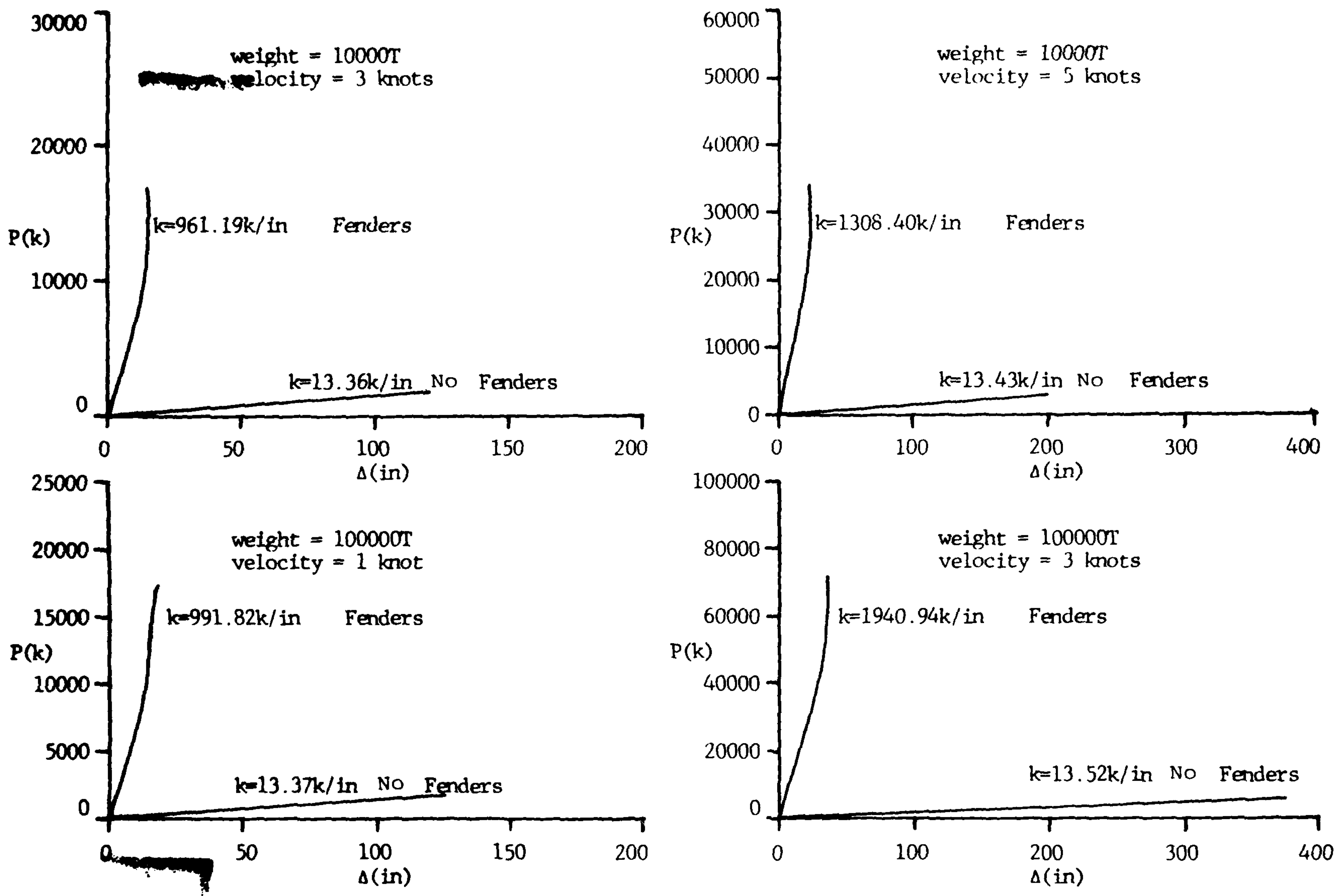


FIG. 1-55

Schuylkill River Bridge

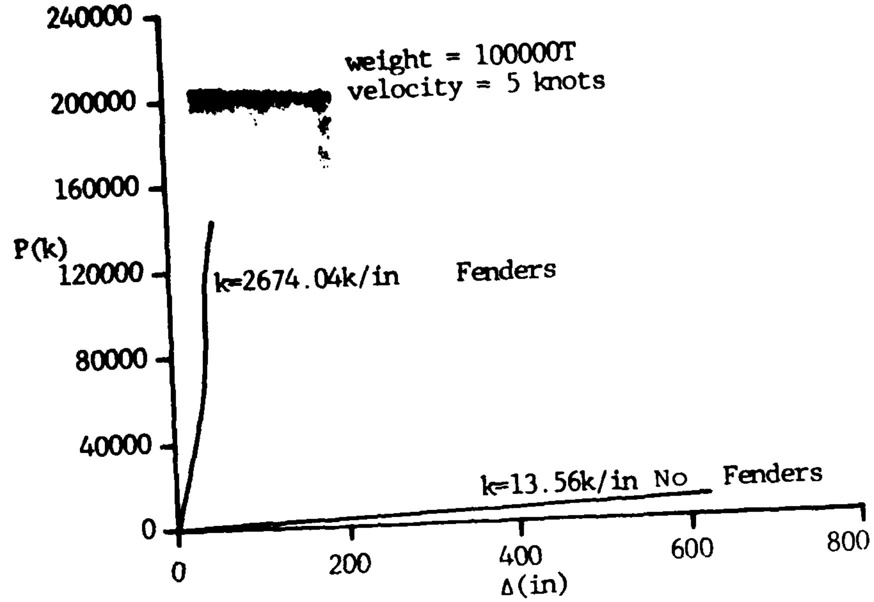


FIG. 1-56

Schuulkill River Bridge

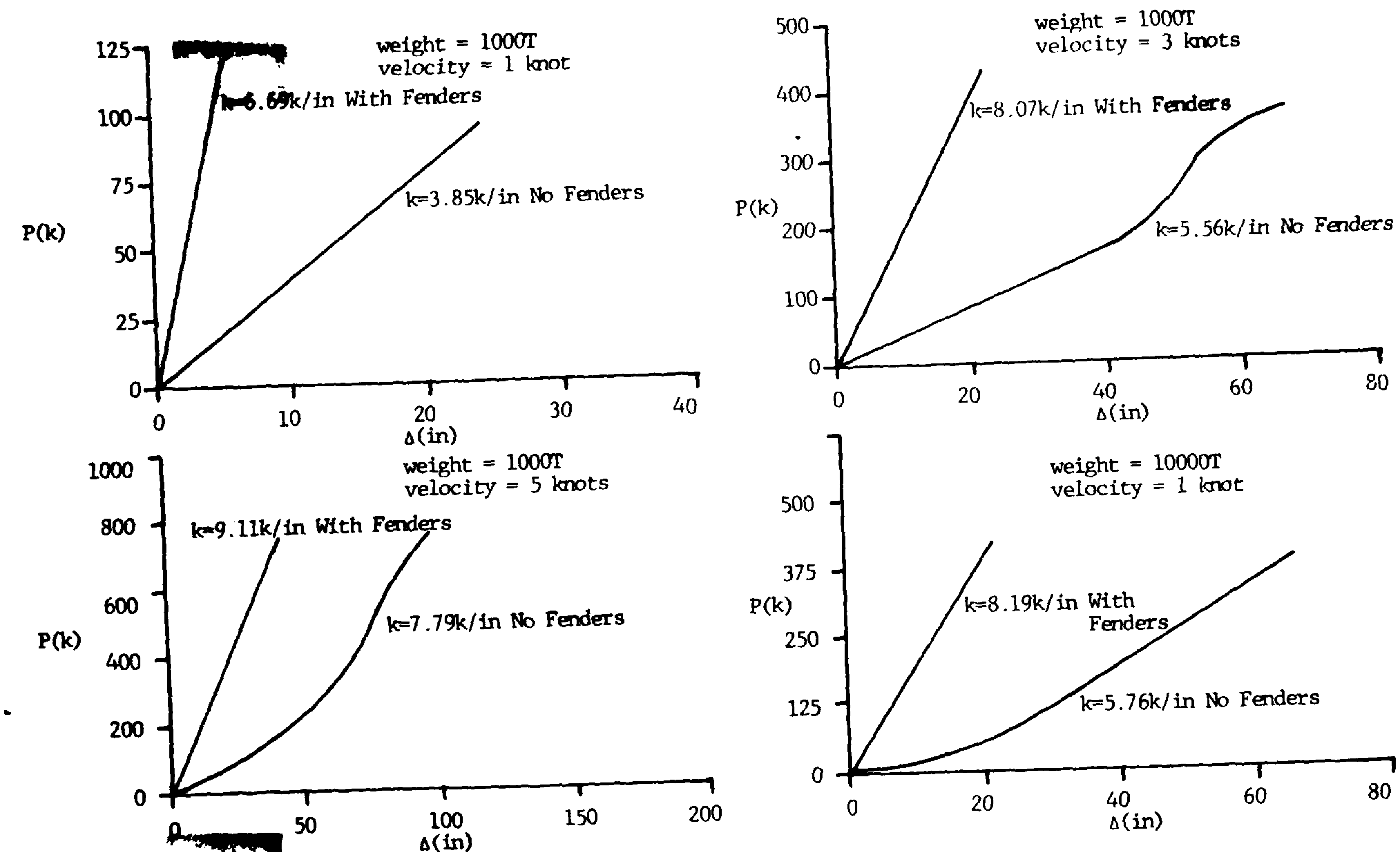


FIG. 1-57

Connecticut River Bridge

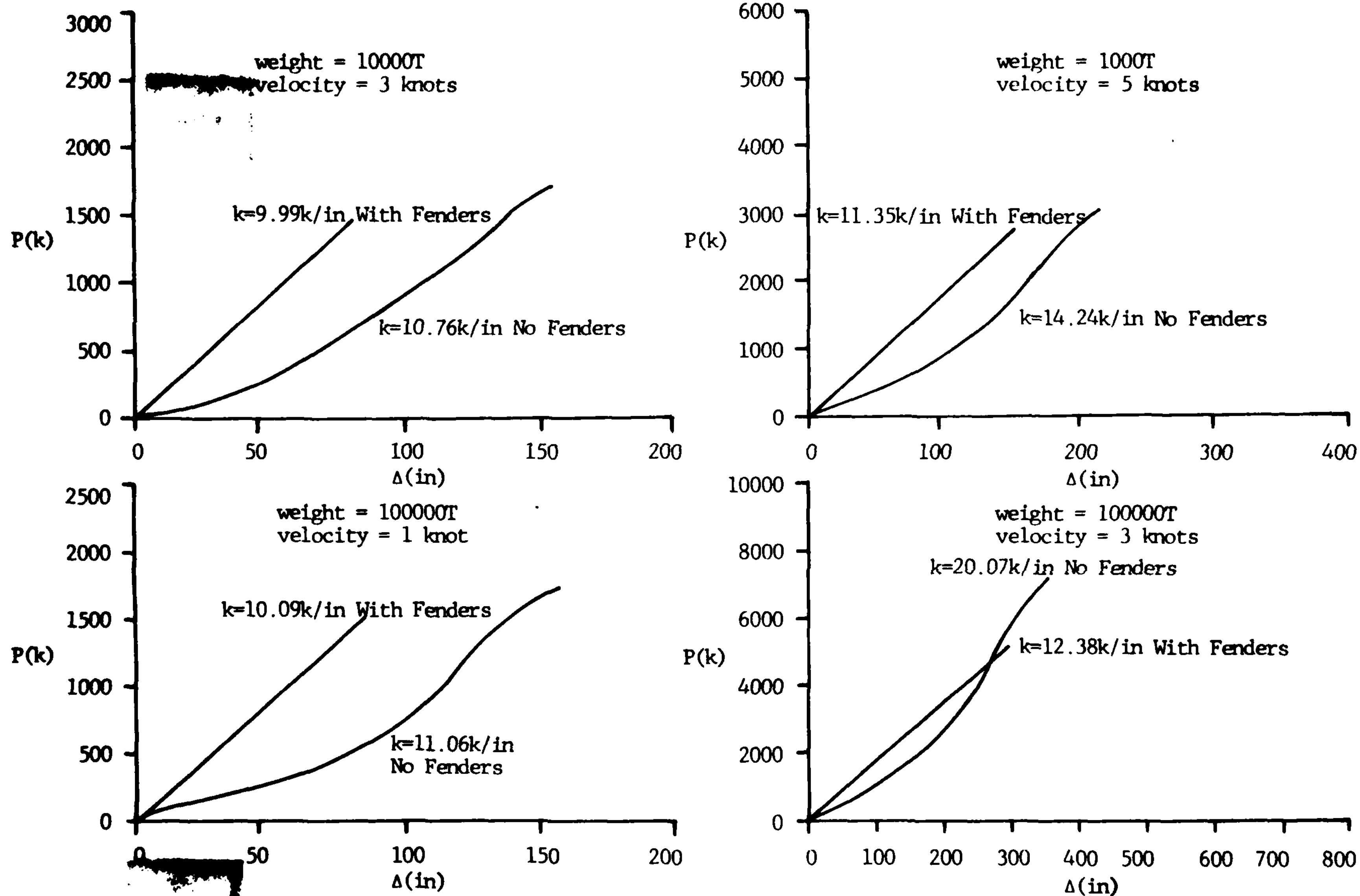


FIG. 1-58

Connecticut River Bridge

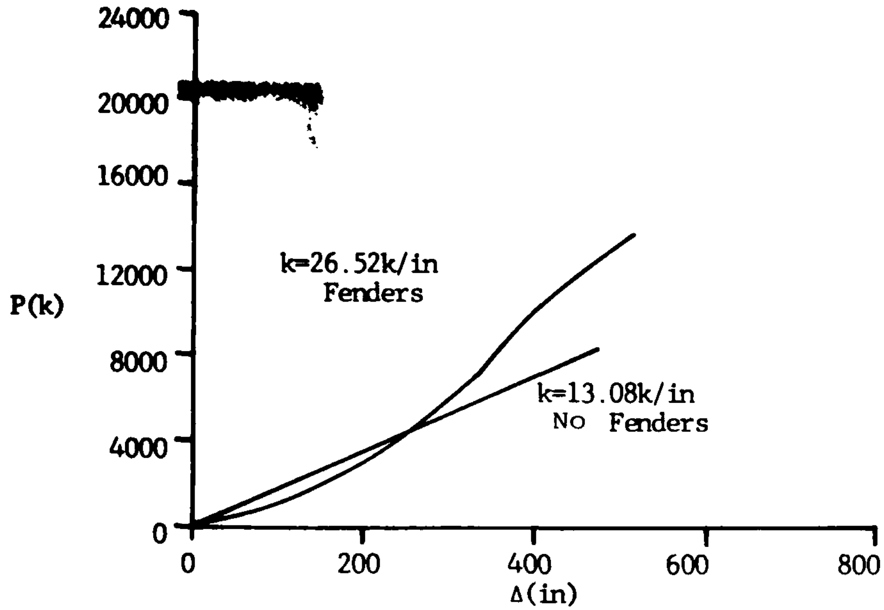


FIG. 1-59

Connecticut River Bridge

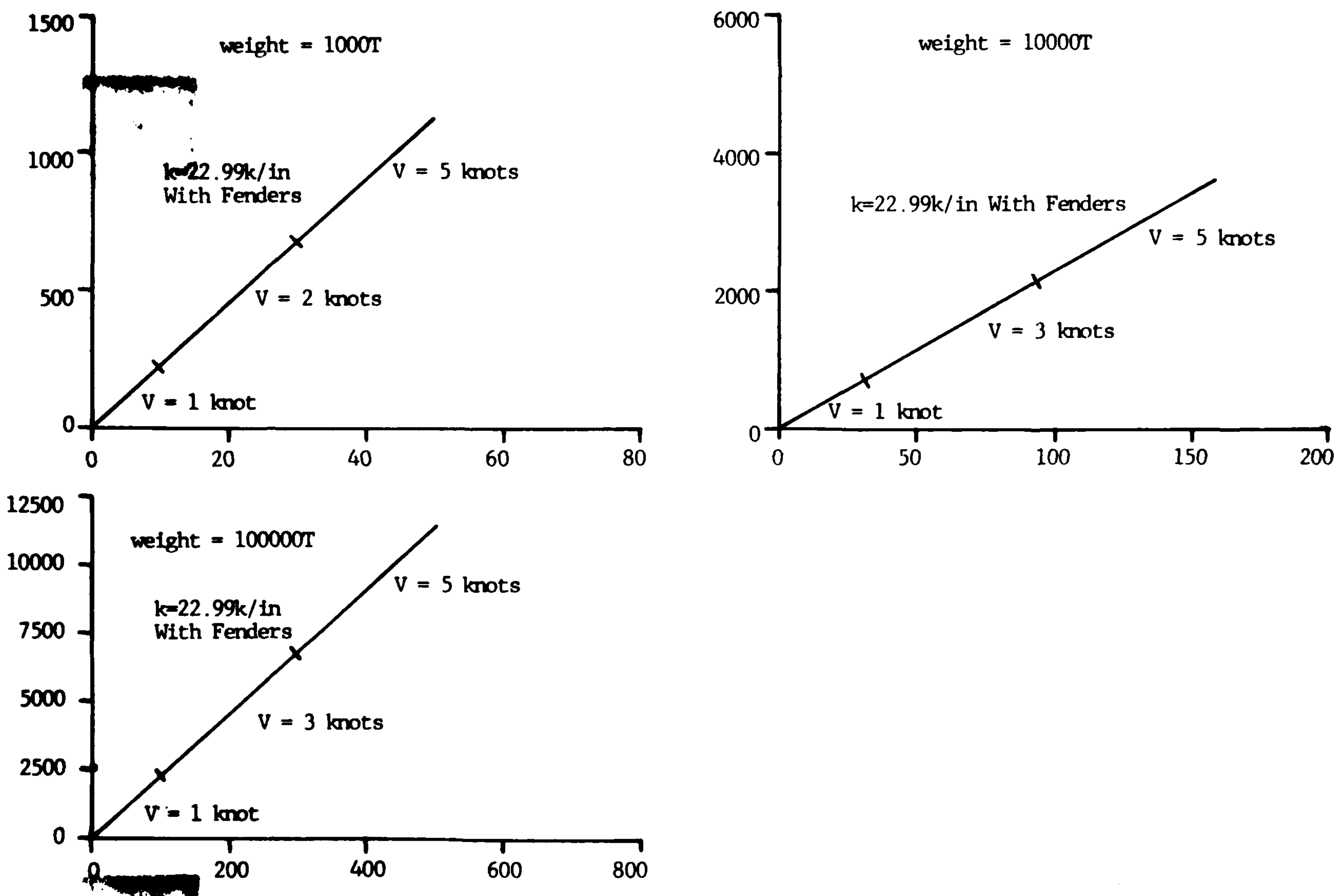


FIG. 1-60

Mare Island Navy Yard

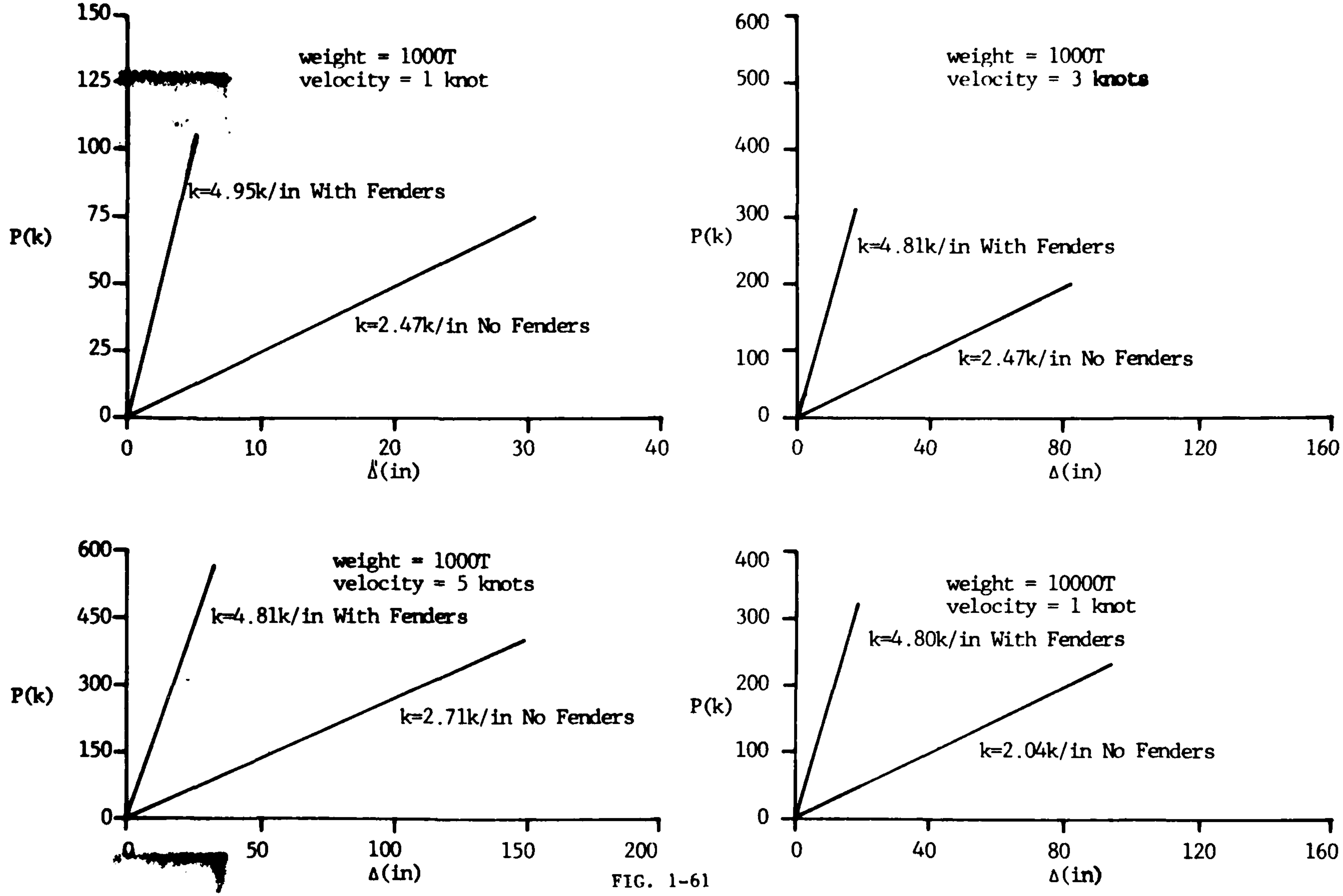


FIG. 1-61

Duwamish Channel Bridge, Skagit Co., Wash. (1)

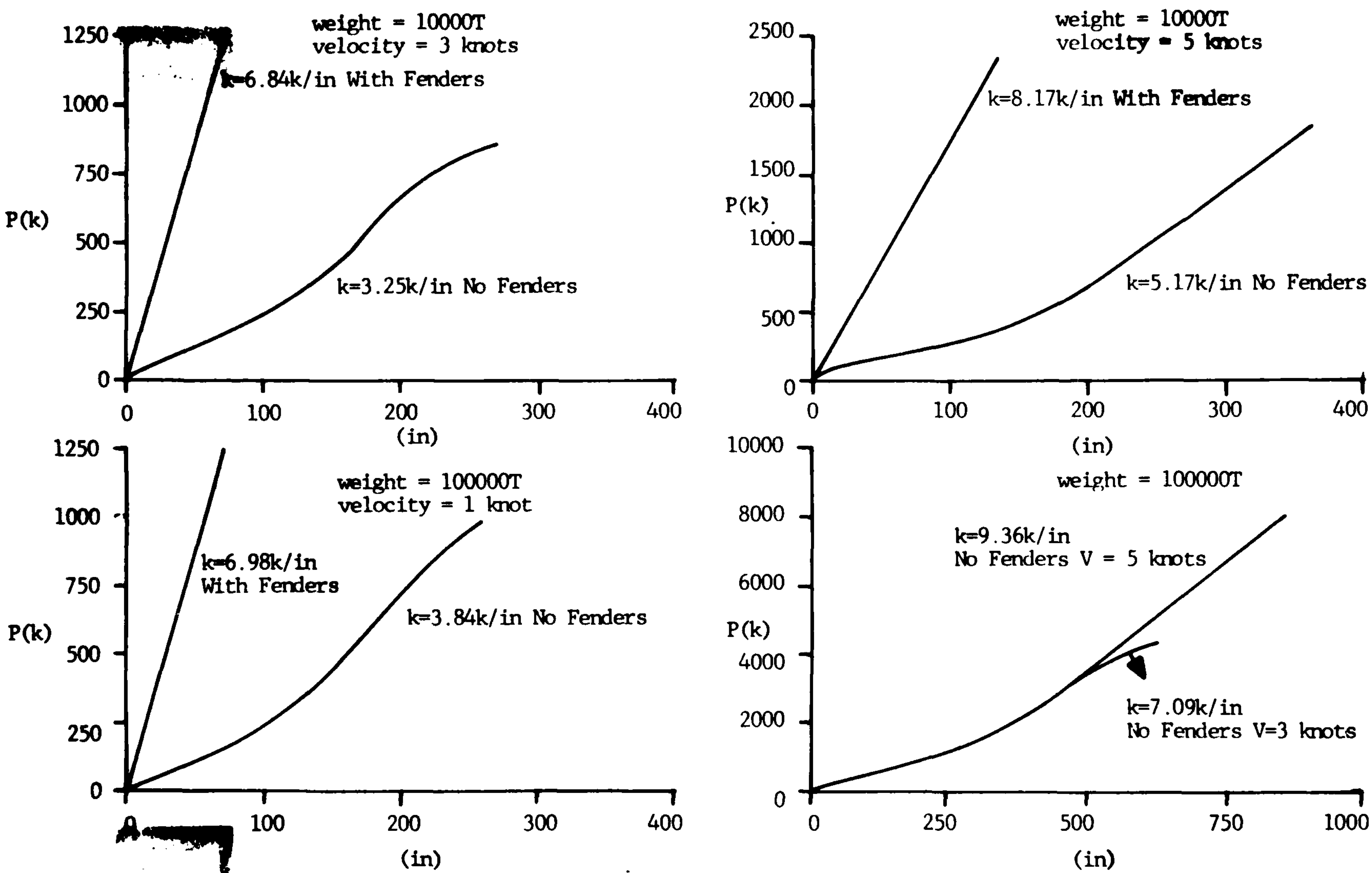


FIG. 1-62

Duwamish Channel Bridge, Skagit Co., Wash. (2)

APPENDIX D

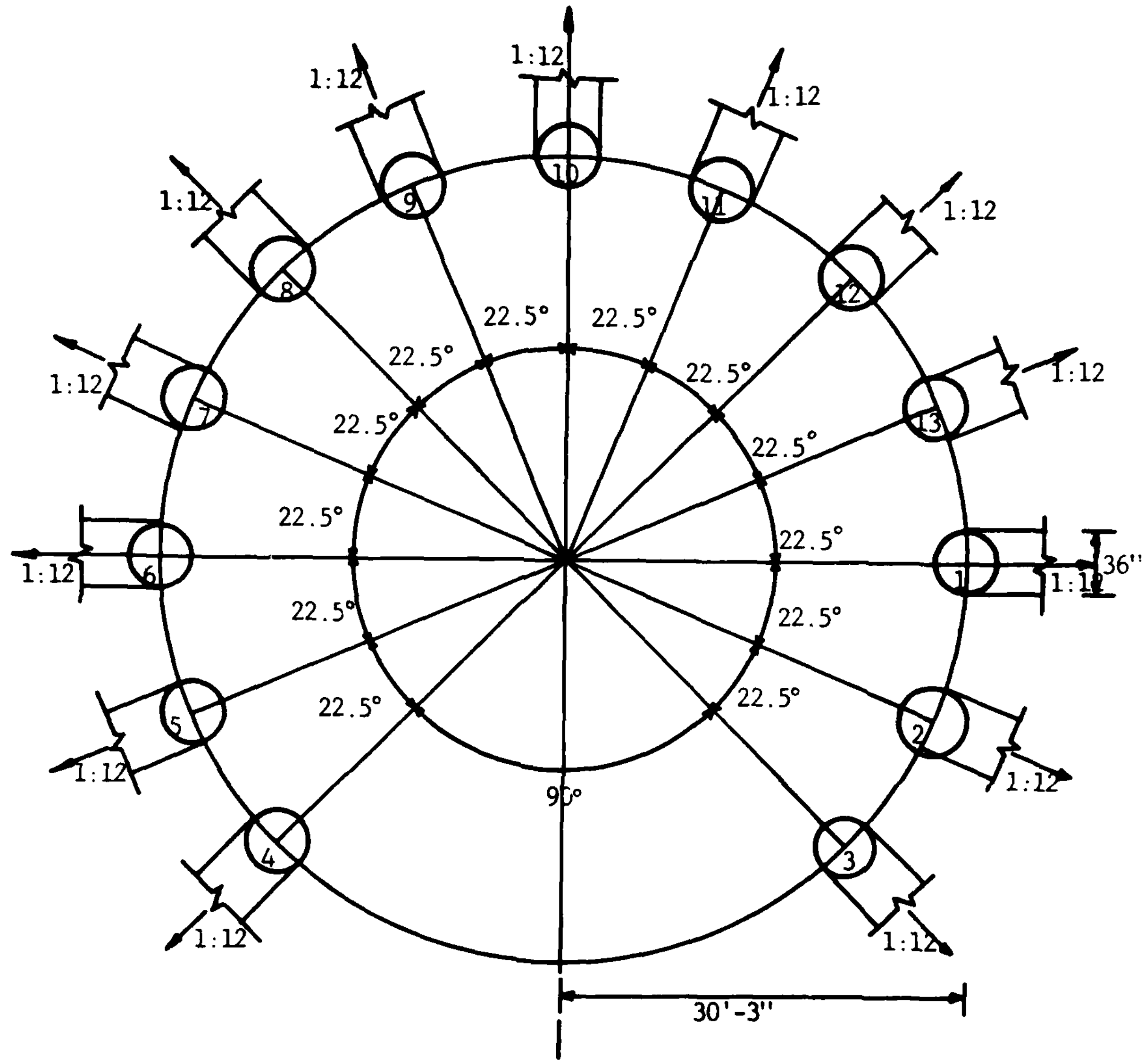
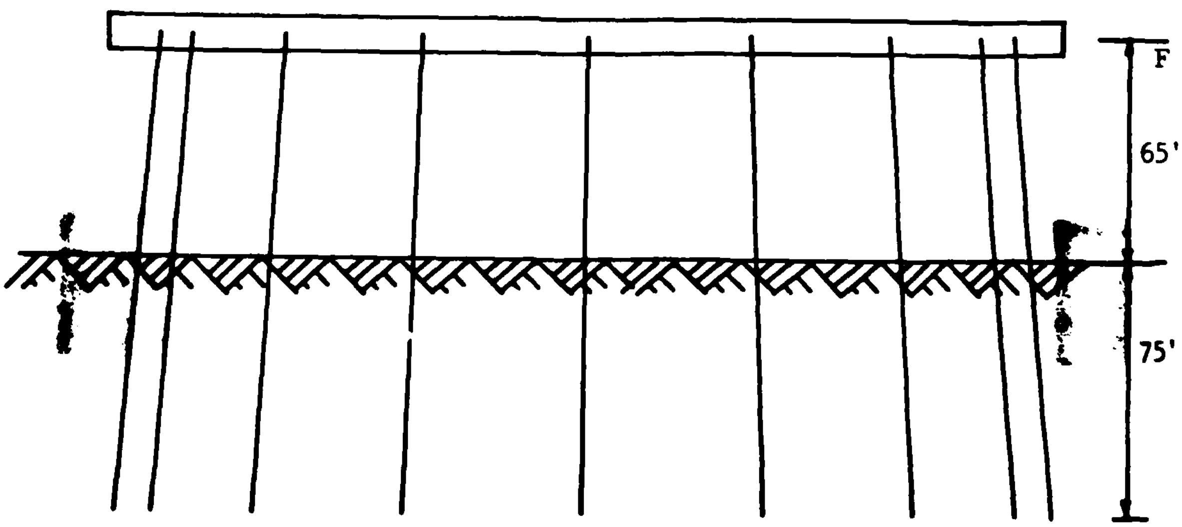


Fig. 2.1: Continuous Round Pile System



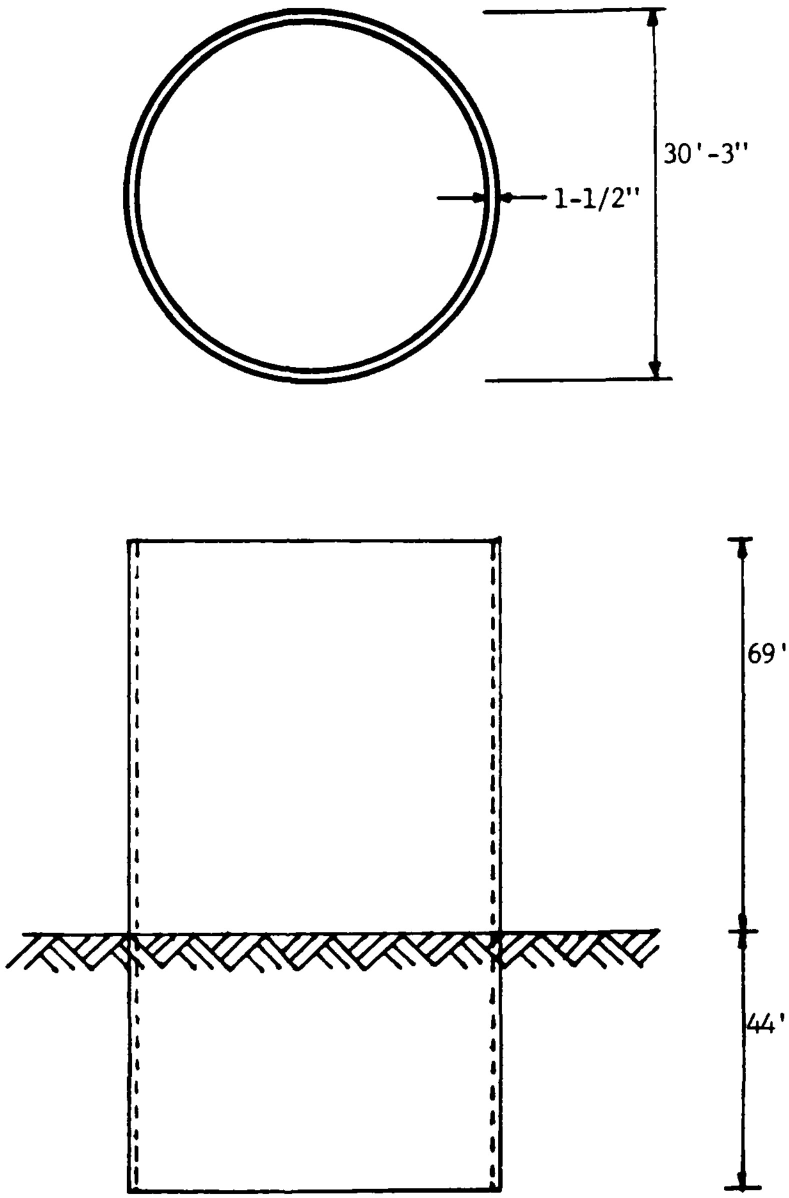


Fig. 2.2: Steel Sheet Pile

BERWICK BAY SKEWED CAP

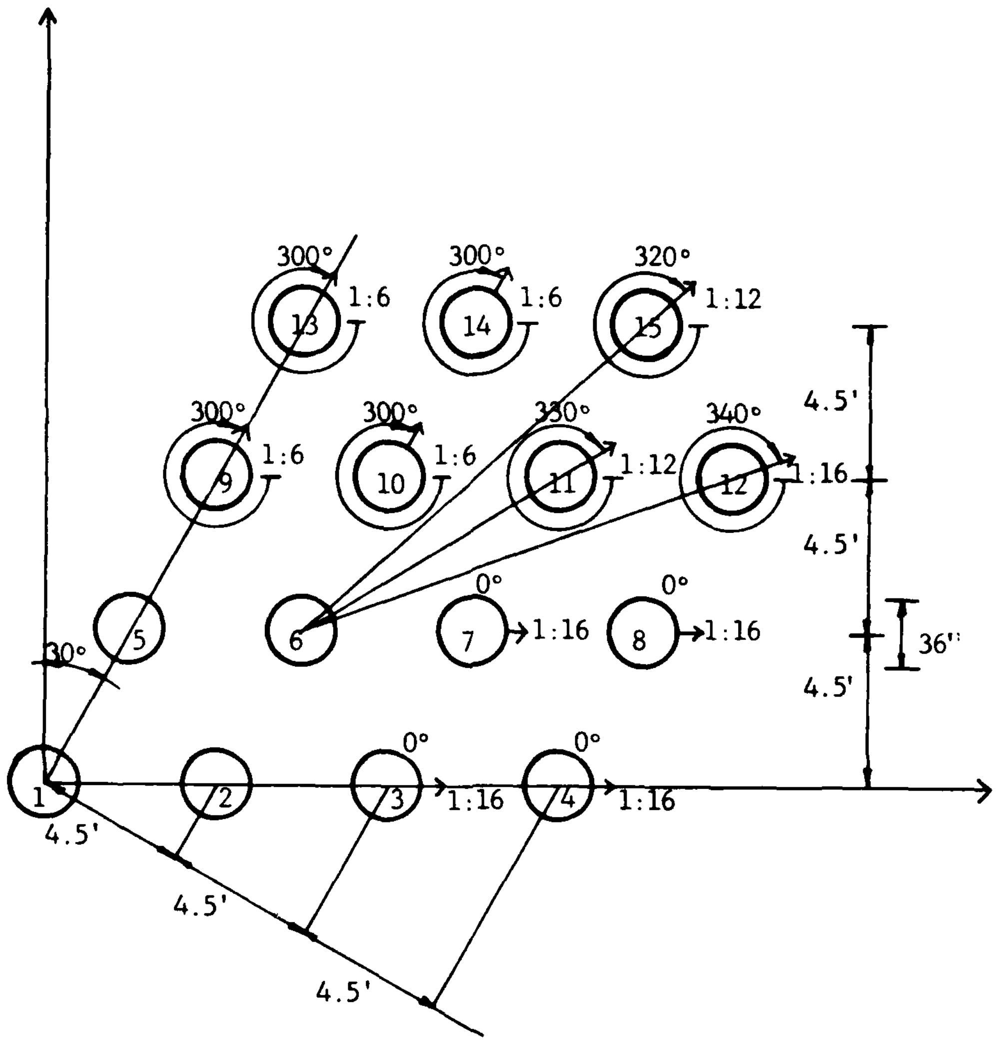
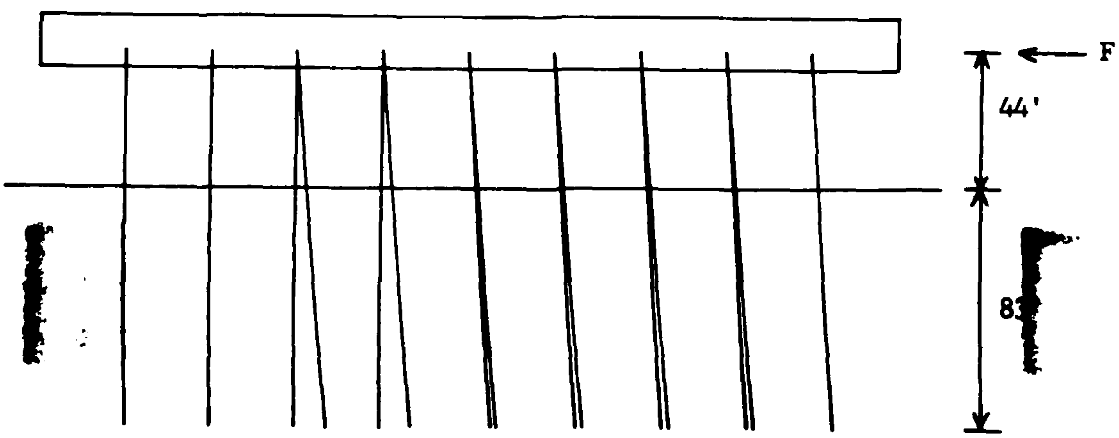


Fig. 2.3: Skewed Shape Cap Pile System



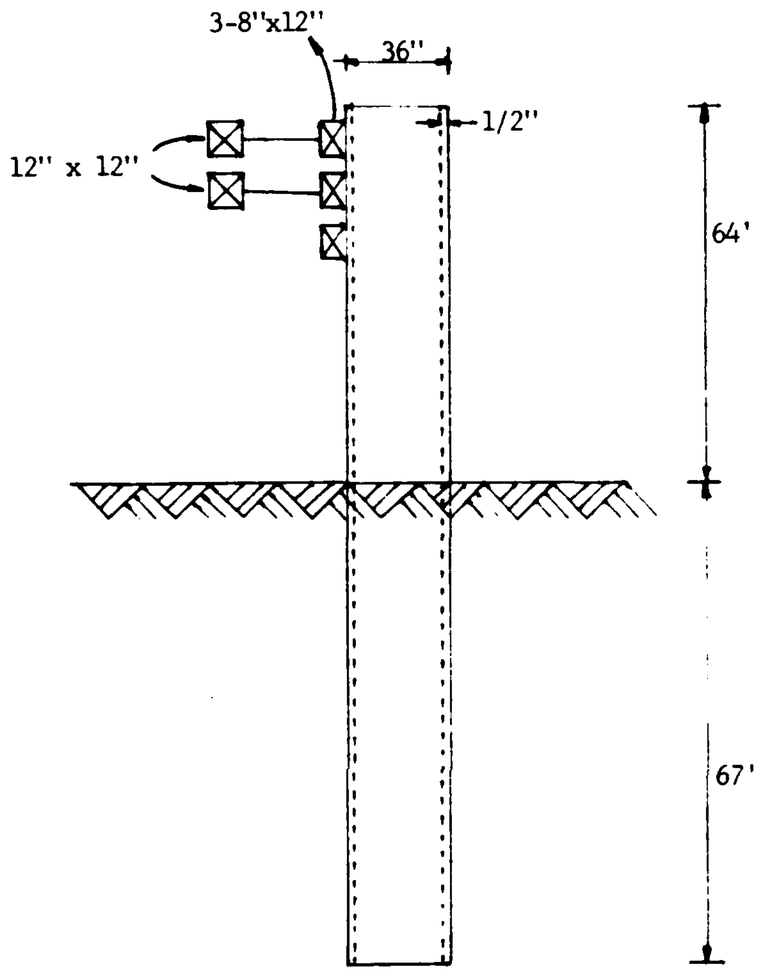
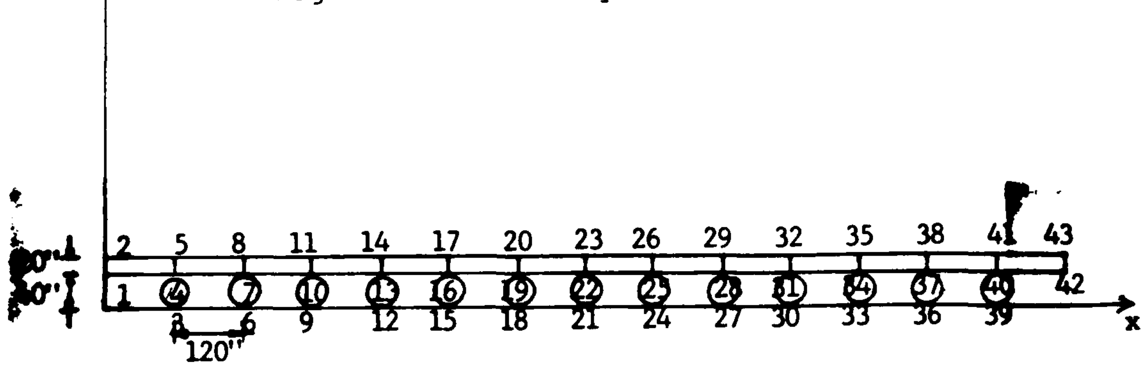


Fig. 2.4: Fender System



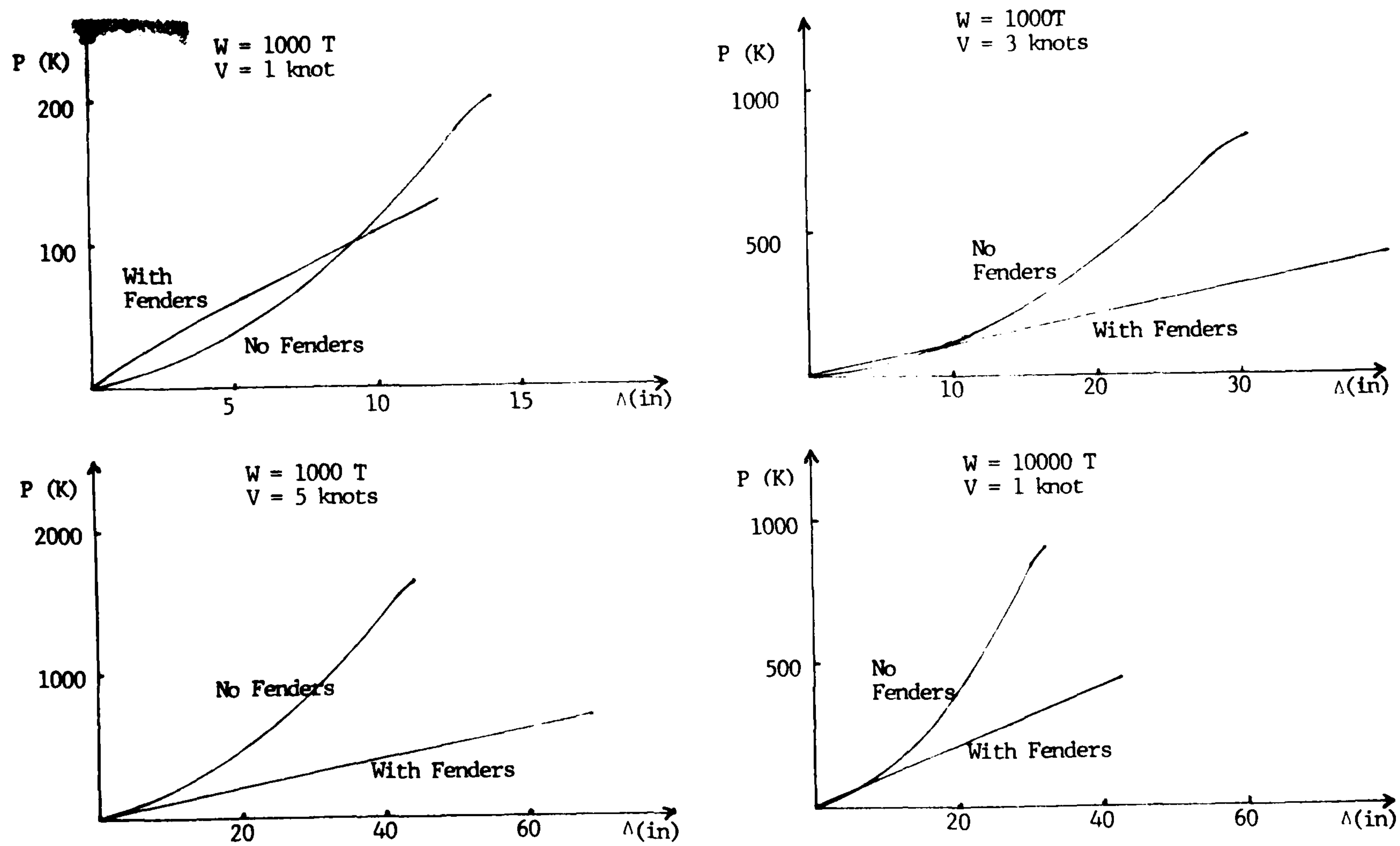


Fig. 2.5: Continuous Round Pile System

BERWICK BAY BRIDGE

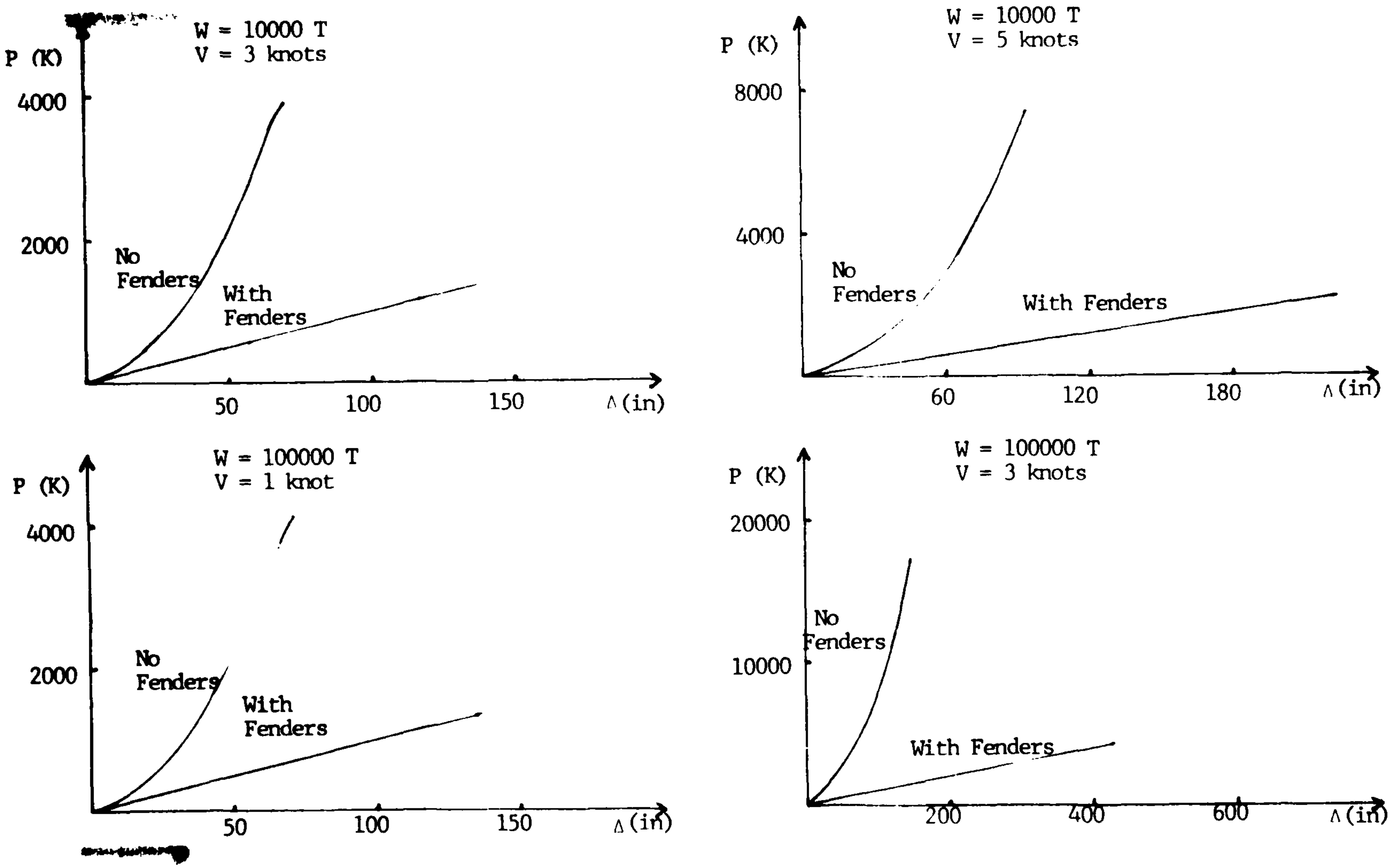


Fig. 2.6: Continuous Round Pile System

BERWICK BAY BRIDGE

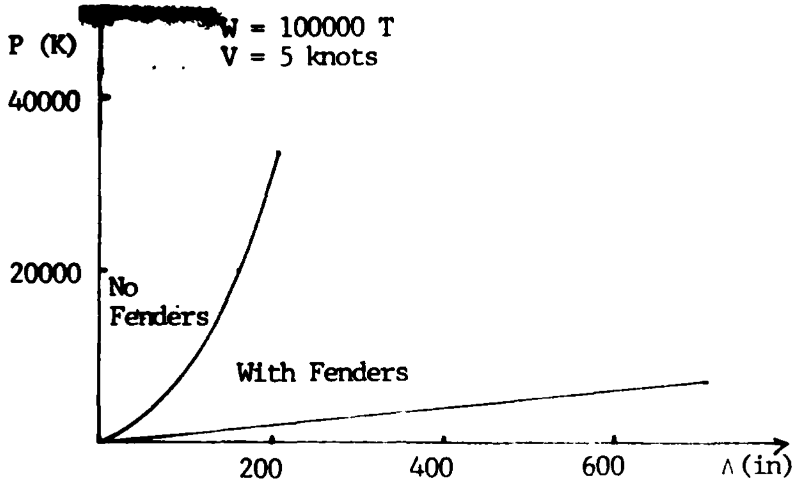


Fig. 2.7: Continuous Round Pile System

BERWICK BAY BRIDGE

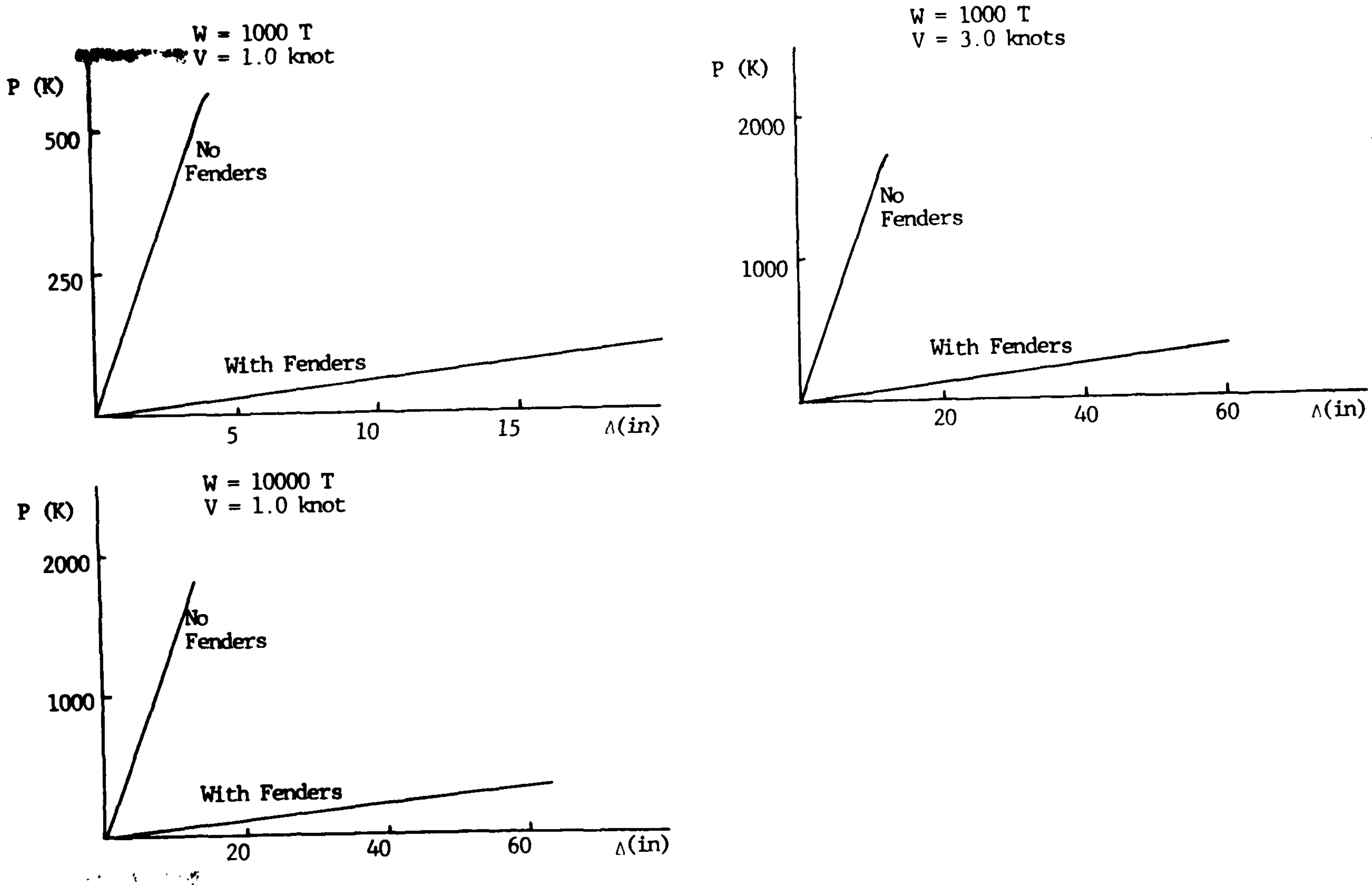


Fig. 2.8: Skewed Cap Pile System
BERWICK BAY BRIDGE

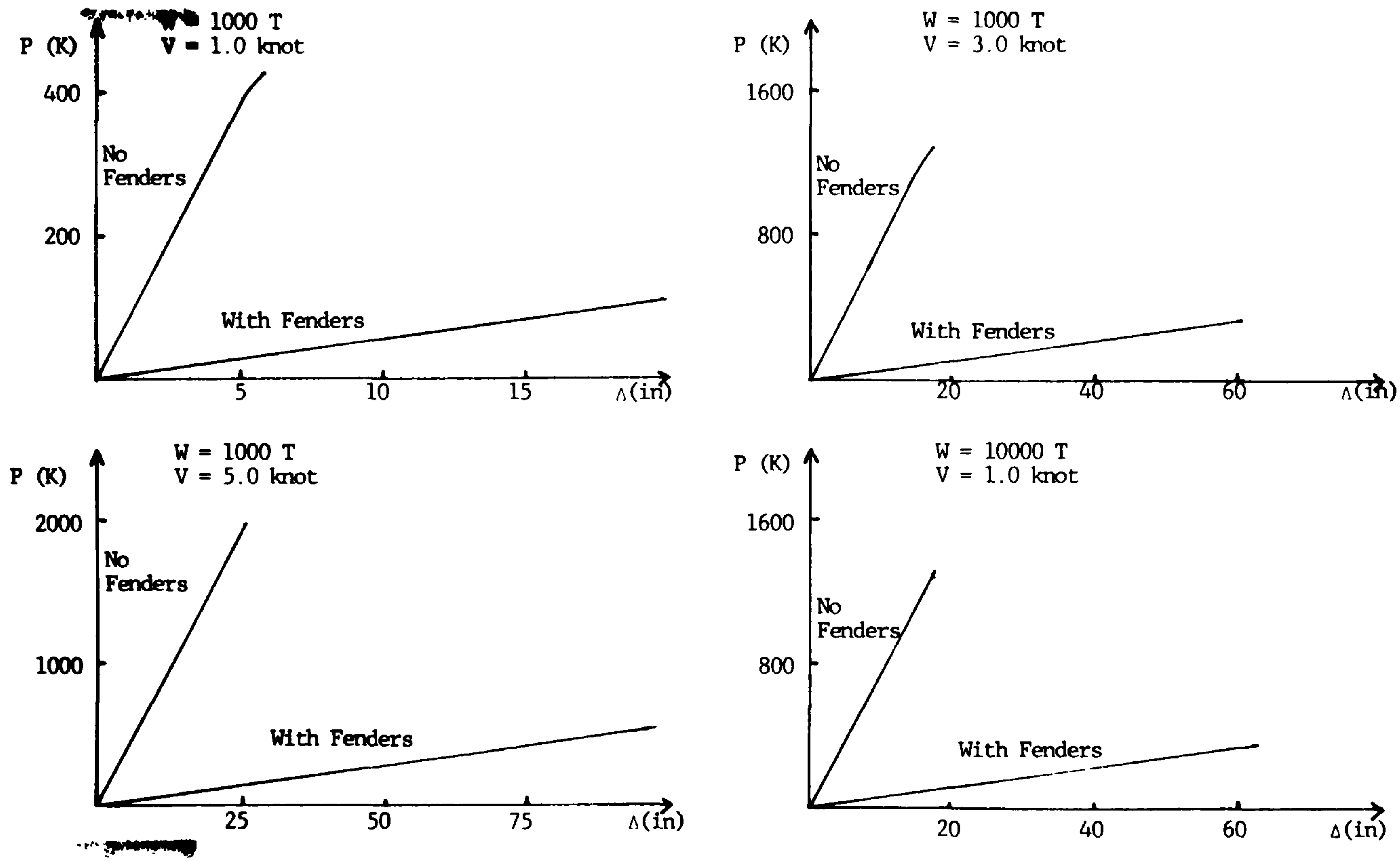


Fig. 2.9: Fender System

TABLE 2.1: Continuous Round Pile System

MODELING OUTLINE

A. Material Properties:

1. Subgrade Modulus - 172.8 k/ft^3
2. Modulus of Rigidity - 11000.0 ksi (Steel Piles)
3. Modulus of Elasticity - 29000.0 ksi (Steel Piles)

B. Soil Properties:

1. Angle of Passive Failure - 45°
2. Dissipation Factor - 0.50
3. Allowable Strain - 0.03 in/in

C. Pile Data:

1. Total length - 140 ft.
2. Contilever length - 65 ft.
3. Circular Pile X & Y projection - 36 in.
4. Pile thickness - 0.391 in.
5. X & Y moment of I - 7164.29 in^4
6. Polar moment of I - 14328.58 in^4
7. Crossectional area - 43.75 in^2
8. Yield Stress - 55.0 ksi
9. Neutral axis to outer fiber length - 18 in.

D. System Data:

1. Structural Configuration - Separated piles with Cap.
2. No. of piles - 13
3. Type of Pile Resistance - Friction
4. Vertical logic - all piles are non-vertical.
5. Fender stiffness - 5.70 k/in.

TABLE 2.2: Steel Sheet Pile

BERWICK BAY STEEL SHEET PILES

MODELING OUTLINE

A. Material Properties:

1. Subgrade Modulus - 172.8 k/ft^3
2. Modulus of Rigidity - 1750.0 ksi (Concrete Mass)
3. Modulus of Elasticity - 4000.0 ksi (Concrete Mass)

B. Soil Properties:

1. Angle of Passive Failure - 45°
2. Dissipation Factor 0.50
3. Allowable Strain - 0.03 in/in

C. Pile Data: (Concrete Mass)

1. Total length - 113 ft.
2. Contilever length - 69 ft.
3. Circular Pile X & Y projection - 363 in.
4. Pile thickness - 0.0 in.
5. X & Y moment of I - 852307664.0 in^4
6. Polar moment of I - 1704615328.0 in^4
7. Crossectional area - 103491.0 in^2
8. f_c' - 2500 psi
9. Neutral axis to outer fiber length - 181.5 in.
10. Fender Stiffness - 5.7 k/in.

TABLE 2.3: Skewed Shape Pile

MODELING OUTLINE

A. Material Properties:

1. Subgrade Modulus - 172.8 k/ft^3
2. Modulus of Rigidity - 11000.0 ksi (Steel Piles)
3. Modulus of Elasticity - 29000.0 ksi (Steel Piles)

B. Soil Properties:

1. Angle of Passive Failure - 45°
2. Dissipation Factor - 0.50
3. Allowable Strain - 0.03 in/in

C. Pile Data

1. Total length - 127 ft.
2. Contilever length - 44 ft.
3. Circular Pile X & Y projection - 36 in.
4. Pile thickness - 0.391 in.
5. X & Y moment of I 7164.29 in^4
6. Polar moment of I - 14328.58 in^4
7. Crossectional area - 43.75 in^2
8. Yield Stress - 55.0 ksi
9. Neutral axis to outer fiber length - 18 in.

D. System Data:

1. Structional configuration - seperated piles with cap
2. No. of piles - 15
3. Type of Pile Resistance - Friction
4. Vertical logic - some piles are non-vertical
5. Fender stiffness - 5.70 k/in

TABLE 2.4: FENDER SYSTEM

Berwick Bay Bridge - Modeling Outline

A. Material Properties:

1. Subgrade Modulus - 175.00 k/ft^3
2. Modulus of Rigidity - 11500.0 ksi (Steel Piles)
3. Modulus of Elasticity - 29000.0 ksi (Steel Piles)

B. Pile Data (one vertical pile)

1. Total Length - 131 ft.
2. Catilever Length - 64 ft.
3. Circular Pile X & Y projection - 36 in.
4. X & Y moment of I - 8787.0 in^4
5. Polar moment of I - 17574.0 in^4
6. Spacing between Piles - .0 in.
7. Crossectional area - 55.76 in^2
8. Yield Stress - 36.0 ksi
9. Neutral axis to outer fiber length - 18 in.
10. Vertical load - 0.0

C. System Data

1. Pile groups - 13
2. Fendr Stiffness - a = 0.02, b = -0.39, c = 5.70
3. Spacing between pile groups - 120 in.
4. Support Beam Modulus of Elasticity - 1200 ksi (Timber)
5. Support Beam moment of I - 3 ¶ 512 = 1530 in^4
6. Support Beam Area - 3 ¶ 96 = 288 in^2
7. Fender Beam Modulus of Elasticity - 1200 ksi (Timber)
8. Fender Beam moment of I - 2 ¶ 1728 = 3456 in^4
9. Fender Beam area - 2 ¶ 144 = 288 in^2

APPENDIX E

HOPEWELL BRIDGE - MODELING OUTLINE

A. Material Properties:

1. Subgrade Modulus - 175.00 k/ft^3
2. Modulus of Rigidity - 75.0 ksi (Timber Piles)
3. Modulus of Elasticity - 1200 ksi (Timber Piles)

B. Pile Data (2 vertical piles)

1. Total length - 50 feet
2. Cantilever length - 25 feet
3. Circular pile X and Y projection - 12 inches
4. X and Y moment of I- 1010 in^4
5. Polar moment of I - 2036 in^4
6. Spacing between piles - 24 inches
7. Crossectional area - 113.0 in^2
8. Yield stress - 5 ksi (Timber Piles)
9. Neutral axis to outer fiber length - 6 inches
10. Vertical load - 0.0

C. System Data

1. Pile groups - 11
2. Fender stiffness - a= 0.089 b= -4.58 c= 76.32
3. Spacing between pile groups - 48 inches
4. Support Beam Modulus of Elasticity - 1200 ksi (Timber)
5. Support beam moment of I - 5 at 216 = 1080 in^4
6. Support Beam Area - 5 at 72 = 360 in^2
7. Fender Beam Modulus of Elasticity - 1200 ksi (Timber)
8. Fender Beam moment of I - 5 at 216 = 1080 in^4
9. Fender Beam Area - 5 at 72 = 360 in^2

TABLE 3.1: Modeling Detail

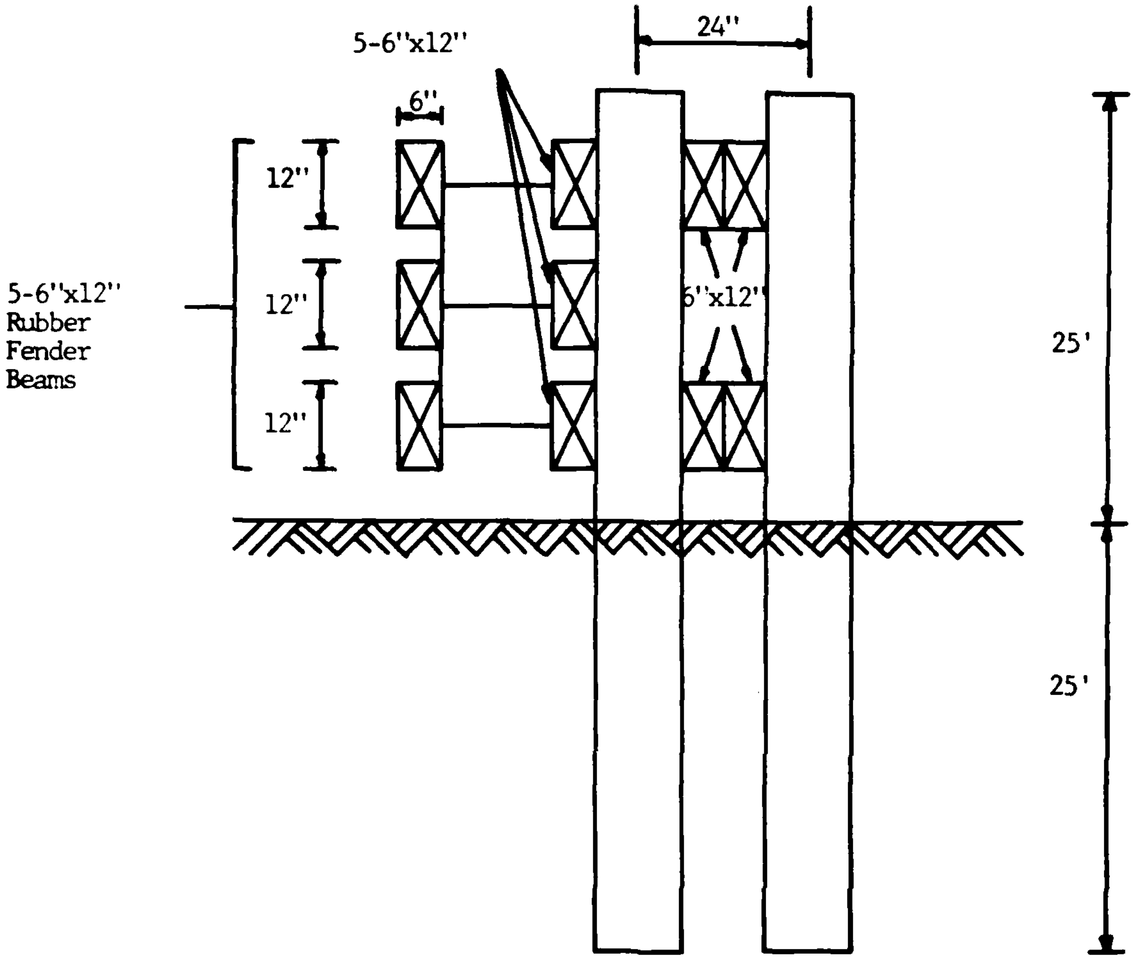
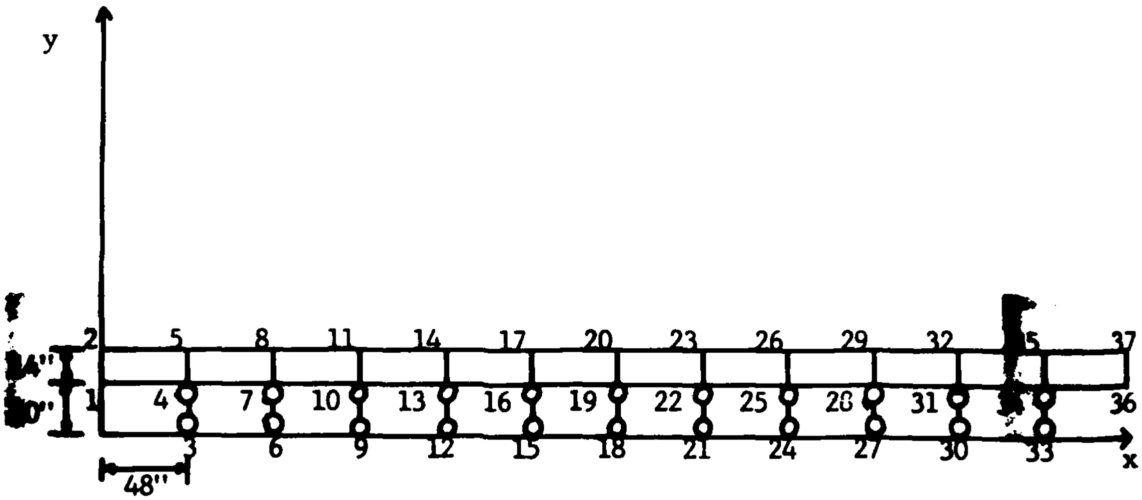
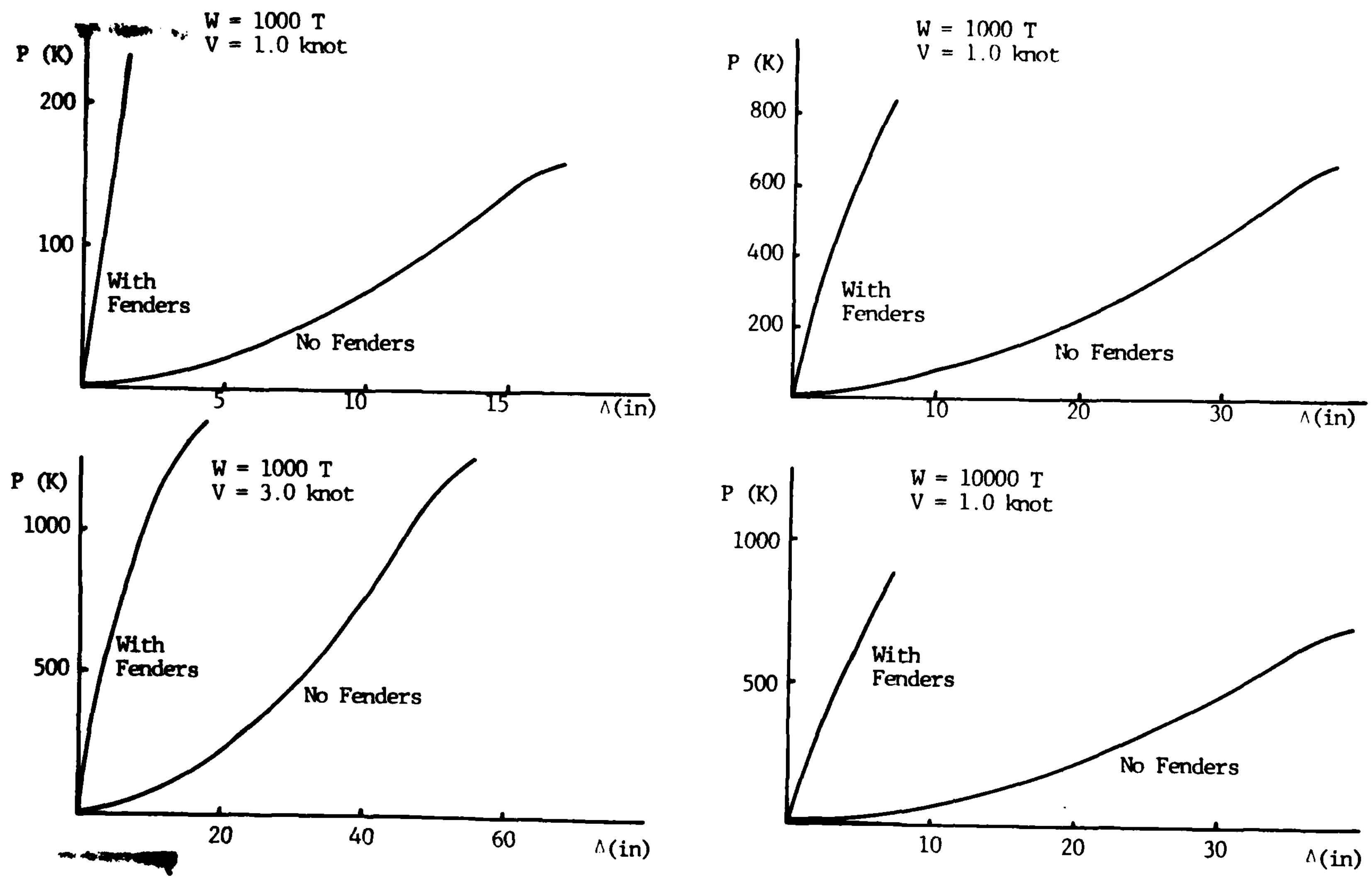


Fig. 3.1a: Modeling Details



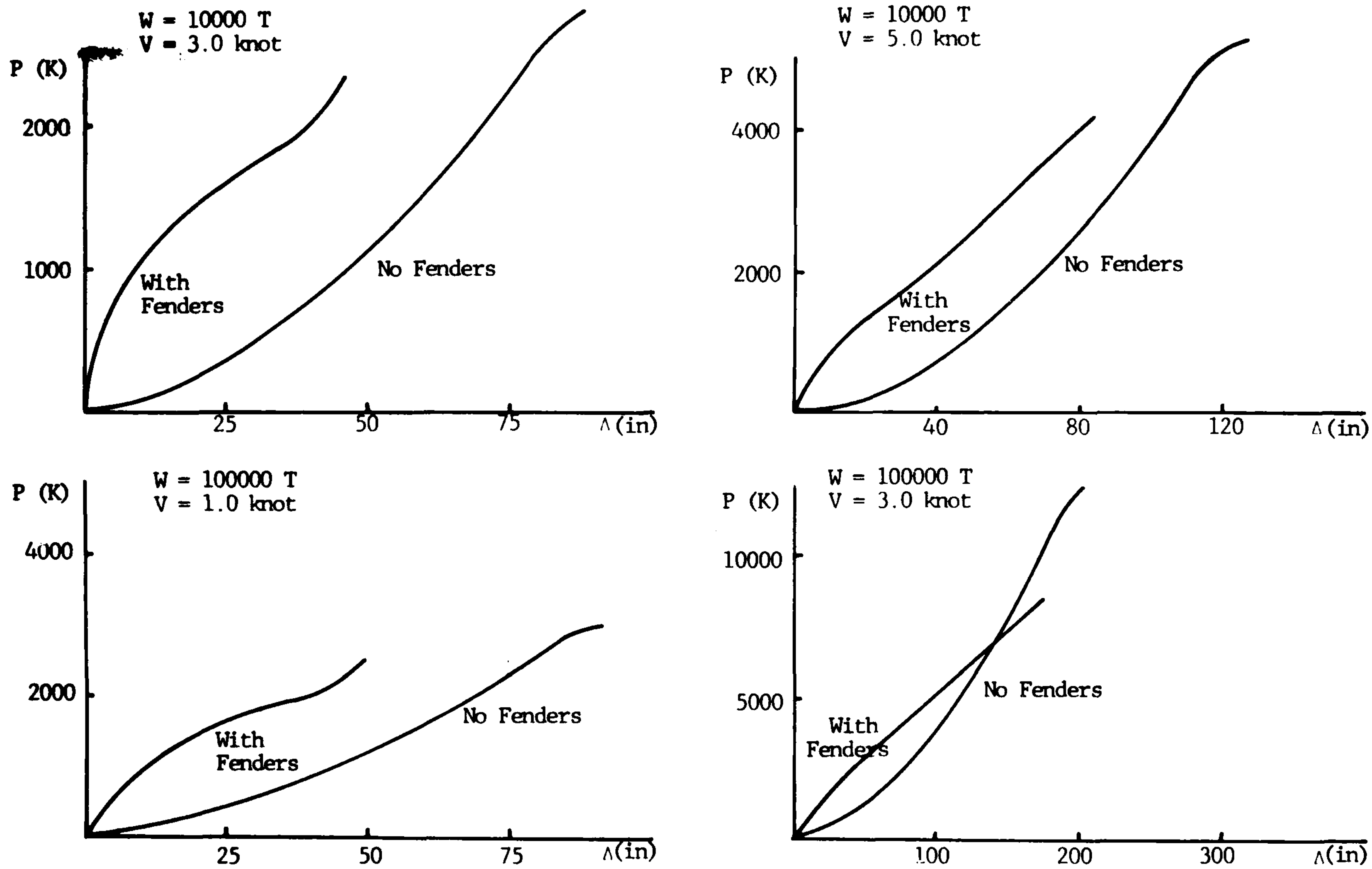
HOPEWELL

Fig. 3.1b: Structural Details



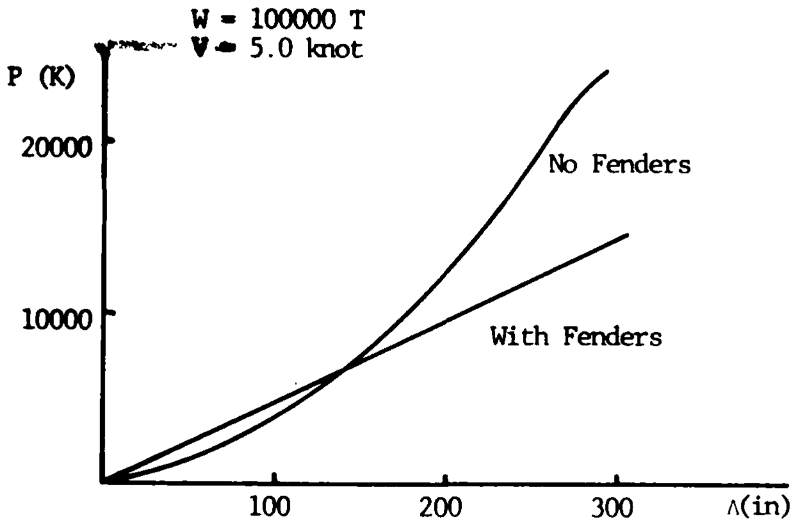
HOPEWELL BRIDGE

Fig. 3.2: Load vs. Defl.



HOPEWELL BRIDGE

Fig. 3.2: Load vs. Defl.



HOPEWELL BRIDGE

Fig. 3.2: Load vs. Defl.

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