Benchmark Ultimate Strength Analysis - Sample Application of API RP 2A, Section 17

Prepared for

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&
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This document was prepared under joint sponsorship of the Minerals Management Service (MMS) and the American Petroleum Institute (API) with technical direction provided by the API Task Group 10 (formerly called T6 92-5).

In December 1996, the API issued Supplement No. 1 to the Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms (API RP2A) [1] which included new guidelines for the assessment of existing structures. The guidelines, located in Section 17 of the API RP2A Supplement, include a consequence based assessment methodology which is significantly different from conventional design. An example of these differences is the use of nonlinear, ultimate strength, analysis to estimate the actual response of a platform to extreme metocean loadings as compared to linear elastic analysis with explicit and implicit safety factors as applied in design.

Prior studies that have applied the new assessment criteria [4] have indicated that there is significant potential for variation in the analysis results depending upon the interpretation of the Section 17 guidelines as well as the specific modeling and analysis procedures. These studies have shown that lack of familiarity with the assessment process is a significant contributor to variability in results. This report has been developed to provide an illustrative example of the structural modeling and application of Section 17 that can be used by engineers to become more familiar with the process and which also verifies software accuracy. This report compliments the guidelines and the commentary included in the Supplement as well as the technical papers that were written (and are shown as references in the Supplement) to provide background regarding the development of the guidelines.

The sample application which is provided in this report consists of a single Gulf of Mexico drilling/production platform. The example platform includes a 4-leg jacket in a water depth of 157 ft. The report includes all of the information that would be needed to perform a complete assessment of this platform (e.g., site conditions and physical conditions of the platform). The report also includes a description of an analysis methodology as well as the results which are generated from this methodology when using PMB's CAP computer software. It is not the intent of this document to endorse this particular analysis procedure or software or to make claims regarding the benefit of a particular method or software over any other.

This document does not provide a comprehensive discussion of analysis issues that one may encounter with other applications of the Section 17 guidelines. It is not the intent of this document to imply that the modeling and analytical procedures described herein are in any way generically applicable to any set of offshore platforms. This document addresses a number of modeling and analysis issues that are pertinent to this specific example. All other assessment problems (e.g., an 8 leg platform) will involve other modeling and analysis issues that are not discussed in this document. Therefore, the procedures discussed in this document must be considered as a example only and may not be suitable for any other application of the Section 17 guideline.

In addition to analysis results generated by PMB, a series of load and capacity data are also provided that indicate the range of results that are considered, by the API Task Group, to be within the accuracy consistent with the basis of Section 17. These other results were generated from the "Trials/Benchmark JIP" [4,5,6] in which the example problem was analyzed by 13 different organizations.

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1.1 Section 17 Recommended Procedure for Platform Assessment

Section 17, which includes a recommended procedure for assessing existing platforms (Figure 1-1), has been released as part of Supplement No. 1 to the API RP 2A, 20th edition [1, 3]. Section 17 provides guidelines for performing a fitness-for-purpose assessment of steel jacket platforms based on their consequences of failure. It recommends a multi-stage assessment procedure for platforms in U.S. waters, and the use of more sophisticated structural analysis methods to determine the strength of platforms and their acceptability.

The recommended procedure involves design level and ultimate strength analyses. The ultimate strength analysis reduces conservatism and attempts to provide mean estimates of platform system (global) capacities using the best estimates of individual component (local) stiffnesses and capacities.

The loads (such as wave loading) used in an ultimate strength analysis are higher than those used in the Section 17 design level analysis. The loads used in a Section 17 design level analysis are generally lower than those per Section 2 of API RP 2A for new structures.

API Task Group 92-5, responsible for Section 17, developed the acceptance criteria by considering the historical experience of a large number of platforms subjected to extreme loading during the past 40 years. In general, less stringent criteria was recommended for older structures based on the consequences of loss of life and environmental pollution as a result of their failure. Specific economic risk from platform loss to the owner was not included in the criteria development. [2]

Due to this new approach to platform assessment, two Joint Industry Projects (JIPs) were undertaken during 1994 to perform trial applications of Section 17 to existing offshore platforms by interested operating companies and contractors [4, 5, 6]. The purpose of these JIPs was to test the validity of the Section 17 criteria/procedure and to identify problems in its application to platforms in U.S. waters and in other regions.

The first JIP (Trials JIP) included the analyses of 23 platforms. Site-specific criteria were developed by some participating companies, who applied the Section 17 procedure to platforms in offshore regions outside of the United States, such as the Bay of Campeche, the North Sea, and Offshore Cameroon [5,6]. The second JIP (Benchmark JIP) involved the analysis of a single "benchmark" platform by 13 organizations to compare nonlinear ultimate strength analysis results and establish variabilities [4,5].

This report summarizes the findings of the Benchmark JIP and provides platform details, criteria, and capacity analysis results.

1.2 Background of the Benchmark JIP

The Benchmark JIP completed in December 1994 [4,5] provided a comparison of results from the nonlinear analyses performed by several organizations on a single "benchmark" platform. The variation in these results is a key component to the Section 17 assessment process.

For this JIP, participants independently selected their analysis parameters (metocean parameters, number of directions for analysis, pile-soil strengths, etc.) based upon the platform information, Section 17 criteria, and API RP guidelines. This provided an opportunity to study the range of metocean choices and wind/wave/current loads for a particular platform. This was particularly interesting since a major revision to the API RP 2A wave force calculation methodology was made in the current 20th edition.

Metocean Loads: The wave heights, current speeds, and other parameters selected by some participants differed from the applicable API Section 2 and Section 17 criteria, resulting in significant differences in the lateral loads. These inconsistencies were a result of interpretational differences of the API guidelines, as well as variations in the participants' computer softwares and structural models.

As a result of this outcome, the API Task Group (TG) performed a follow-up investigation with the voluntary cooperation of eleven of the organizations involved in the Benchmark JIP. These participants were provided with the "correct" metocean criteria and procedures applicable to the benchmark platform. Using the given wave height, wave period, current, and wind data, the participants provided revised loading estimates to the TG. [5]

Ultimate Capacity: A total of thirteen companies performed the ultimate capacity analyses of the benchmark platform. Nine different nonlinear capacity analysis software packages were utilized, which represented the majority of such software available in the offshore industry. The variations in results were found in: (1) capacities at first element failure, (2) failure modes and mechanisms, and (3) ultimate capacities.

Variabilities in Results: The following variabilities were ultimately determined from the JIP and the TG investigations:

Sample No.	Item	COV
1	Total lateral loading (11 participants)	**0.07**
	— Wind loading	0.31
	— Wave & current on jacket	0.09
	— Wave & current on deck	0.76
2	Ultimate capacity (13 participants)	**0.23**
3	Ultimate capacity (10 participants) (#1)	**0.16**

(#1): excludes one very high and two lower capacity estimates

The above COVs in loading (7%) and capacity (16%) estimates determined in the JIP are considered by the API Task Group to be within a range of "acceptable" results. These results confirmed the TG's assumptions used in developing the acceptance criteria regarding variability of loading and ultimate strength estimates. Not reflected in these estimates are the inherent uncertainties in the loading and capacity formulations, and the deviations (biases) in analytical estimates from the actual behavior of platforms.

1.3 Objectives of this Document

The primary objective of this document is to provide a sample application of the Section 17 guidelines in determining the lateral and overturning static ultimate strength of a steel jacket platform in an extreme storm or hurricane event. The documented results will provide an "acceptable" basis to verify analysis procedures, models, and results of analyses performed on other platforms.

The following information is provided in this document:

- Platform and geotechnical data
- Metocean data
- Modeling assumptions
- Load analysis results
- Capacity analysis results
- Variabilities in the loading and capacity estimates provided by Benchmark JIP participants

The capacity analysis results are provided for two cases: (1) with complete soil-structure interaction included; and (2) with fixity at the pilehead to exclude the effect of soil-structure interaction.

The range of the JIP results highlight the variations to be expected in the estimate values when different criterias, modeling techniques, analysis procedures and software are applied.

1.4 Use of this Document

This document is intended to provide a sample application of the ultimate strength analysis procedures recommended in Section 17 for evaluation of the global integrity and survivability of platforms against large storms. In this sample application, the dynamic effect is ignored and the structural members are considered to have no damage or deterioration.

This document is not intended to provide the only procedure for evaluating the ultimate capacity of steel jacket platforms. Various other alternative procedures are suggested in the Section 17 document, and additional procedures may emerge in later years. The appropriate approach to the analysis of an individual platform (e.g., number of storm directions, modeling refinements, analysis refinements) will depend upon its characteristics, its existing condition, and its likely state during its remaining life, and therefore requires engineering judgment.

The sample analysis represents a process that is applicable to a specific structure. Other types of analysis may be more applicable to other structures in assessing their particular characteristics, such as dynamic analysis, joint behavior, etc. It is the responsibility of the user to understand the specific requirements of a particular structure in the context of the Section 17 objectives.

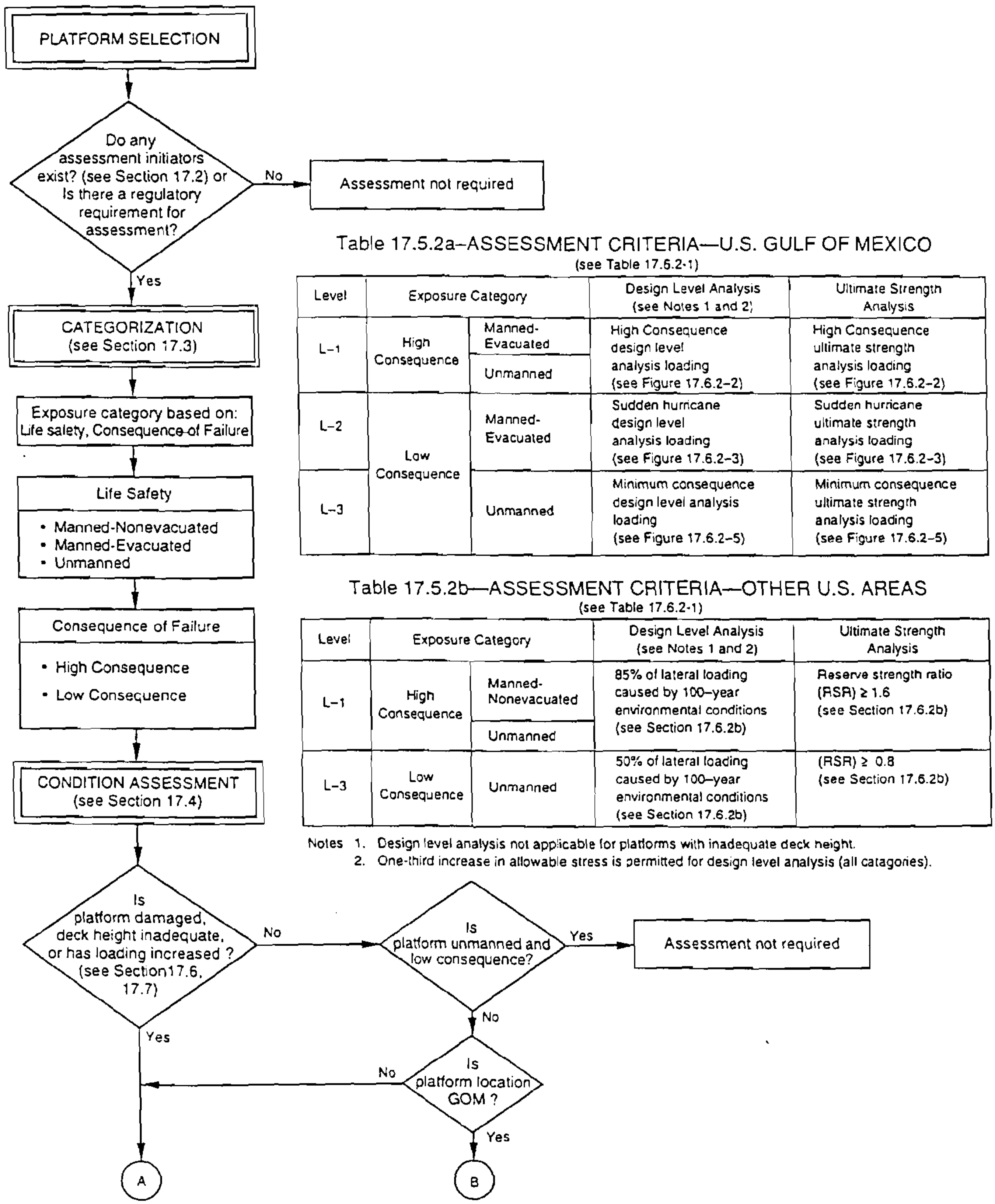


Table 17.5.2a–ASSESSMENT CRITERIA—U.S. GULF OF MEXICO
(see Table 17.6.2-1)

Level	Exposure Category		Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
L-1	High Consequence	Manned-Evacuated	High Consequence design level analysis loading (see Figure 17.6.2-2)	High Consequence ultimate strength analysis loading (see Figure 17.6.2-2)
		Unmanned		
L-2	Low Consequence	Manned-Evacuated	Sudden hurricane design level analysis loading (see Figure 17.6.2-3)	Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2-3)
L-3		Unmanned	Minimum consequence design level analysis loading (see Figure 17.6.2-5)	Minimum consequence ultimate strength analysis loading (see Figure 17.6.2-5)

Table 17.5.2b—ASSESSMENT CRITERIA—OTHER U.S. AREAS
(see Table 17.6.2-1)

Level	Exposure Category		Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
L-1	High Consequence	Manned-Nonevacuated	85% of lateral loading caused by 100–year environmental conditions (see Section 17.6.2b)	Reserve strength ratio (RSR) ≥ 1.6 (see Section 17.6.2b)
		Unmanned		
L-3	Low Consequence	Unmanned	50% of lateral loading caused by 100–year environmental conditions (see Section 17.6.2b)	(RSR) ≥ 0.8 (see Section 17.6.2b)

Notes 1. Design level analysis not applicable for platforms with inadequate deck height.
2. One-third increase in allowable stress is permitted for design level analysis (all catagories).

Figure 1-1 Section 17 Platform Assessment Process - Metocean Loading (Refer Fig. 17.5.2 of Supplement 1 API RP 2A, 20th Ed.)

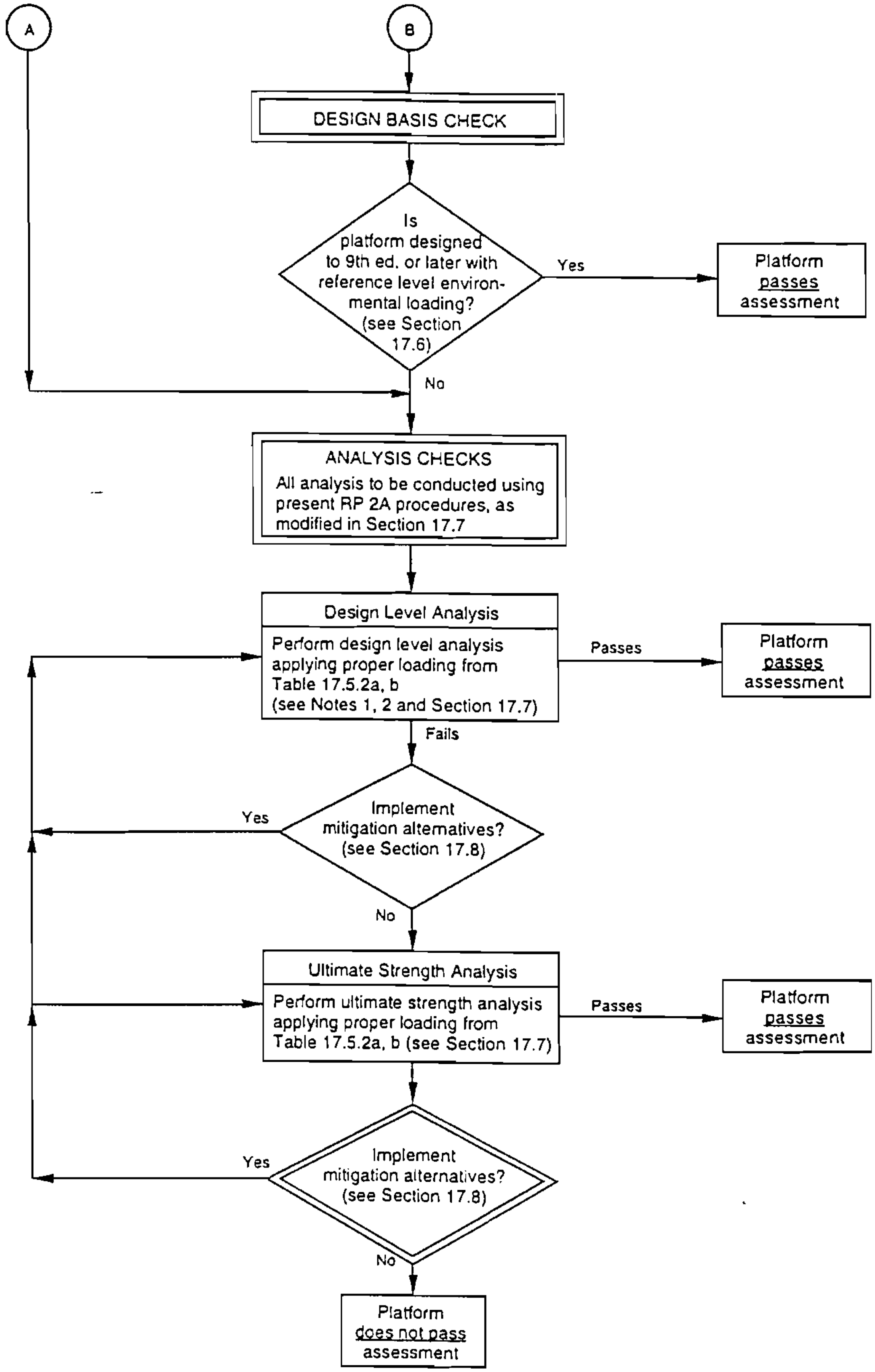


Figure 1-1 Section 17 Platform Assessment Process - Metocean Loading (Refer Fig. 17.5.2 of Supplement 1 API RP 2A, 20th Ed.) [Continued]

The benchmark platform is an existing, 4-leg, 4-pile structure typical of platforms found in the Gulf of Mexico. Located in the Ship Shoal area of the Gulf of Mexico, the platform was installed in 1970, in water depth of 157 feet. The platform is oriented Eastward, 45 degrees from True North (Figure 2-1), and is located at latitude 28° 27′ and longitude 91° 20′.

Details required to perform an ultimate capacity analysis of the platform are provided in this section.

2.1 Deck

The deck structure consists of two bi-level decks (lower and upper decks) installed separately and spliced together at Elevation (+)49′-6″. The elevations of the four levels of the complete deck structure vary from (+)33′ to (+)71'-4 1/8", and include facilities to support four production wells, five risers, a quarters building, and other equipment.

The lower deck consists of the first and second deck levels, and the upper deck consists of the third and fourth deck levels. The lower deck is connected to the jacket/pile at Elevation (+)16′. The spacing between the legs is 30 feet in both orthogonal directions.

Modeling of the deck structure in this example is not intended for the assessment of the deck capacity, but rather to simulate the effect of the deck (i.e. weight, stiffness, etc.) on the jacket.

2.1.1 Structural Details

The primary details of the deck structure are given in Figures 2-2 to 2-7.

The sizes of selected members of the deck structure are:

Lower deck structure:

Deck legs	36" dia x 1.25″ thick tubulars
Deck truss braces	10.75″ dia and 12.75″ dia tubulars
Truss upper chord	21WF62 girder
Truss bottom chord	12.75″ dia tubular
Second deck framing - secondary girders	W14x17.2 at 3 ft spacing
Second deck plating	1/4-inch chequered plate

Upper deck structure:

Deck legs	36″ dia x 1.0″ thick tubulars 36″ dia x 0.625″ thick tubulars
Third level (lower) girders	21WF55
Fourth level (upper) girders	36WF150
Knee braces - fourth level deck	8.625″ and 12.75″ dia tubulars
Third deck framing - secondary girders	14WF34
- plating	1/4-inch chequered plate
Fourth deck framing - secondary girders	16WF36
- plating	1/4-inch chequered plate

The structural details of the stairs from the upper deck to the lower deck are given in Figure 2-8, and the details of the stairs from the lower deck to the jacket top horizontal framing are given in Figure 2-9.

2.1.2 Equipment Data

The overall floor plan, equipment layout, and the estimates of total dead and live loads at the four levels of the deck are given in Figure 2-10.

The primary equipment at the four deck levels is summarized as follows:

Deck Level	Deck Elevation (ft)	Primary Equipment	Equipment Density (for metocean load computation)
First	(+) 33	Horizontal runs of risers and a sump tank (4′x8′x4′)	1/3 of projected area
Second	(+) 43	Generator set, header, well heads	Congested
Third	(+) 56	Production equipment	Congested
Fourth	(+) 71.34	Production equipment and 24′x25′x24′ quarters on SE corner	Congested

The combined weight of the upper and lower deck structures is 350 kips. This includes the weight of the deck legs, braces, and horizontal framing at all deck levels from Elevation (+)16′ (jacket to lower deck stabbing point) to T.O.S. Elevation (+)71′-4 1/8″ of the upper deck.

The dead load (including all structural weight) was considered equally distributed on each of the four leg nodes at four levels. The live load distribution at the four legs is given in Figure 2-10.

The total dead and live loads at the four deck levels is tabulated in the table below:

Deck Level	Dead Load (kips)	Live Load (kips)	Total Load (kips)
First	54	132	186
Second	60	172	232
Third	110	480	590
Fourth	94	640	734
Total	**318 (#1)**	**1,424**	**1,742**

(#1): Excludes 32 kips dead load of deck legs from Elevations (+)16′ to (+)33′.

2.1.3 Projected Areas of Deck and Equipment

The projected areas of the deck and equipment for both orthogonal directions, as indicated in Figure 2-11, are used in the estimation of the wind and wave-in-deck forces.

The projected areas are tabulated as follows:

North and South Directions:

Deck Level	B.O.S. Elevation (ft)	Deck Level Height (ft)	Deck Level Width (ft)	Projected Area (ft^2)
First	32.47	9.66	33	105 (#1)
Second	42.13	12.12	33	400
Third	54.25	14.50	46	667
Fourth	68.75	17.25	51	880

(#1) : Includes a blockage factor of 0.33.

East and West Directions:

Deck Level	B.O.S. Elevation (ft)	Deck Level Height (ft)	Deck Level Width (ft)	Projected Area (ft^2)
First	32.47	10.82	33	119 (#2)
Second	43.29	9.79	33	323
Third	53.08	14.17	51.25	726
Fourth	67.25	18.75	59.75	1,120

(#2) : Includes a blockage factor of 0.33.

2.2 Jacket

2.2.1 Structural Components

The jacket template supports the deck structure, four production wells, eight risers, two boat landings, and four barge bumpers. The spacing between the legs in both orthogonal directions is 30 feet at the work point (Elevation (+)16′) and 61.6 feet at the mudline (Elevation (-)157′). Field measurements taken before installation of the upper deck indicated that the spacing between the legs is 29′-11 5/8″ instead of 30′-0″ as shown in the design drawings. Therefore, the spacings shown in the drawings differ for the upper and lower decks. The bottom of the jacket is at Elevation (-)169′. The jacket vertical framing in both orthogonal directions are identical and are given in Figure 2-12.

Jacket legs: The outer diameter of the legs is 39.5 inches. The leg wall thickness is typically 0.50-inches and 0.75-inches, with an increase to 1.25 inches in the upper bay (splash zone) between Elevations (+)13′ and (-)8′. The leg can thickness and length is 1.25 inches and 10 feet, respectively. The legs extend 12 feet below the mudline. The mudline can and leg extension are 40 inches in diameter with a wall thickness of 1.5 inches.

The legs are provided with a double batter of 1 in 11. The leg-pile annulus is ungrouted.

Vertical Frames: The vertical frames are defined with a K-bracing system. The braces vary from 16 inches to 20 inches in diameter; and wall thickness range from 0.375 inches to 0.50 inches. The braces in the upper bay are 16 inches in diameter with a wall thickness of 0.843 inches. A ′clear gap of 2 inches between braces (K-braces, vertical-to-horizontal) is assumed at all joints.

Horizontal Framings: The jacket is provided with eight horizontal framings, as shown in Figures 2-13 and 2-14. Two bracing patterns (diamond and double-K) are provided in these horizontal framings. The frames at Elevations (-)8′ and (-)97′ are provided with diamond bracing patterns and frames at all other elevations are provided with double-K bracing pattern. The diameters of the primary braces vary from 16 inches to 20 inches.

At elevations (-)8′, (-)48′, and (-)97′, no lateral support (guides) of the conductors are provided. The details of conductor guides provided at all other levels are given in Figure 2-12.

No damage or deterioration of the jacket components was considered in the analysis.

2.2.2 Other Components

The following items were considered in the analysis model.

Conductors: Three 30-inch diameter and one 48-inch diameter conductors are supported by the platform (see Figure 2-1). The 30-inch diameter conductors penetrate 285 feet below the

mudline, while the 48-inch diameter conductor penetrate 150 feet. The wall thickness of these conductors is 0.625-inches.

Risers: Eight risers are provided, spanning from Elevation (+)45′ to the mudline. Their diameters are 3.5 inches (1 riser), 4.5 inches (3 risers), and 6.625 inches (4 risers). A wall thickness of 0.375-inches and a corrosion coating of 0.50-inches was assumed for all risers. The risers are laterally supported at the jacket horizontal framing levels from Elevations (+)10′ to (-)126′. The riser clamp weights at different levels were ignored in the benchmark analysis.

Boat landings: Two boat landings are provided, one each on the East and South sides of the platform. The boat landings are from Elevations (+)10′ to (-)2′-11″. The general configuration of the boat landing is given in Figures 2-15 and 2-16.

Barge bumpers: Four barge bumpers are provided, two each on the East and South sides of the platform. The barge bumpers connect to the jacket legs at Elevations (+)10.22′ and (-)7.11′. The general configuration of the barge bumpers is shown in Figure 2-16.

Anodes: Anodes were not in included in the model (i.e. submerged weight, wave and current loads on anodes).

Miscellaneous items: The self weights of various items such as conductor guides, riser clamps, lifting padeyes, handrails, grating and supporting angle/channel sections, and ladders were not considered in the benchmark analysis.

2.3 Pile Foundation

The piles are connected to the jacket at Elevation (+)13′ (top of jacket elevation). The piles are 36-inch diameter tubulars with a maximum wall thickness of 1.875 inches (from the mudline to 80 feet below). As-designed pile elevations were used in the model. The piles penetrate 355 feet below the mudline, with the pile tip elevation at (-) 512′. The pile makeup (see Figure 2-17) from the pile cut-off level (working point elevation) to the pile tip elevation is given as follows:

Elevations from Mean Sea Level (MSL) (ft)	True Length Between Elevations (ft)	Pile Diameter (in)	Pile Wall Thickness (in)
(+)15.500 to (-)130.143	146.84	36	1.250
(-)130.143 to (-)140.061	10	36	1.500
(-)140.061 to (-)239.245	100	36	1.875
(-)239.245 to (-)249.163	10	36	1.500
(-)249.163 to (-)259.080	10	36	1.250
(-)259.080 to (-)278.920	20	36	1.000
(-)278.920 to (-)512.000	235	36	0.875

2.4 Soil

The variations in soil strength parameters with depth given in Figure 2-18 were taken from the McClelland Engineers, Inc. report of September 1969 for the Ship Shoal area. The soil consists of very soft to very stiff gray clay from the mud level to 197 feet below, and stiff to very stiff gray clay from 225 feet to 391 feet.

The intermittent 28-foot layer from 197 feet to 225 feet consists of very dense silty sand layer. Per API RP 2A, the angle of internal friction (ϕ') was assumed as 30 degrees and N_q (dimensionless bearing capacity factor) as 20 for silty sand layers (Strata II and V).

The initial soil modulus for the sand layer was assumed to be as per API RP 2A, 20th Edition. The piles were considered as plugged for computation of pile axial capacity.

The submerged unit weight (γ') of soil for varying depths below mudline is summarized below (from the soil report):

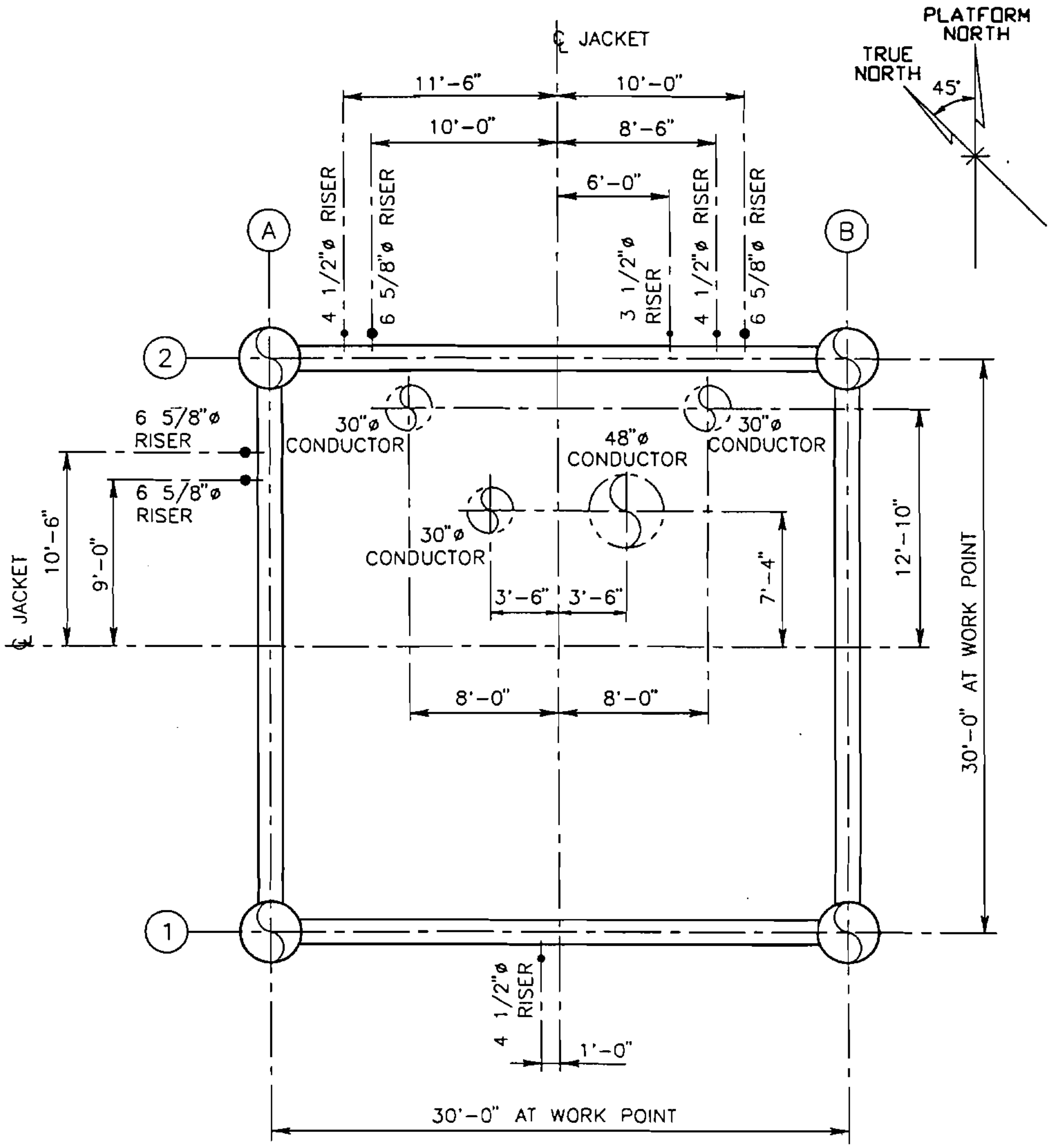
Depth Below Mudline (ft)	Submerged Unit Weight, γ' (k/ft^3)
0 - 100	0.040
100 - 197	0.050
197 - 225	0.060
225 - 391	0.050

The strain at half the maximum deviator stress (ε_c) is as follows (per API RP 2A):

Undrained Shear Strength, S_u (k/ft^2)	Soil Strain, ε_c (%)
0.0 - 0.5	2.0
0.5 - 1.0	1.0
1.0 - 2.0	0.7
2.0 - 4.0	0.5
> 4.0	0.4

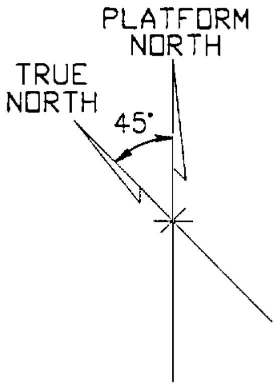
The initial static stiffness parameter (K_s) for stiff clay is as follows:

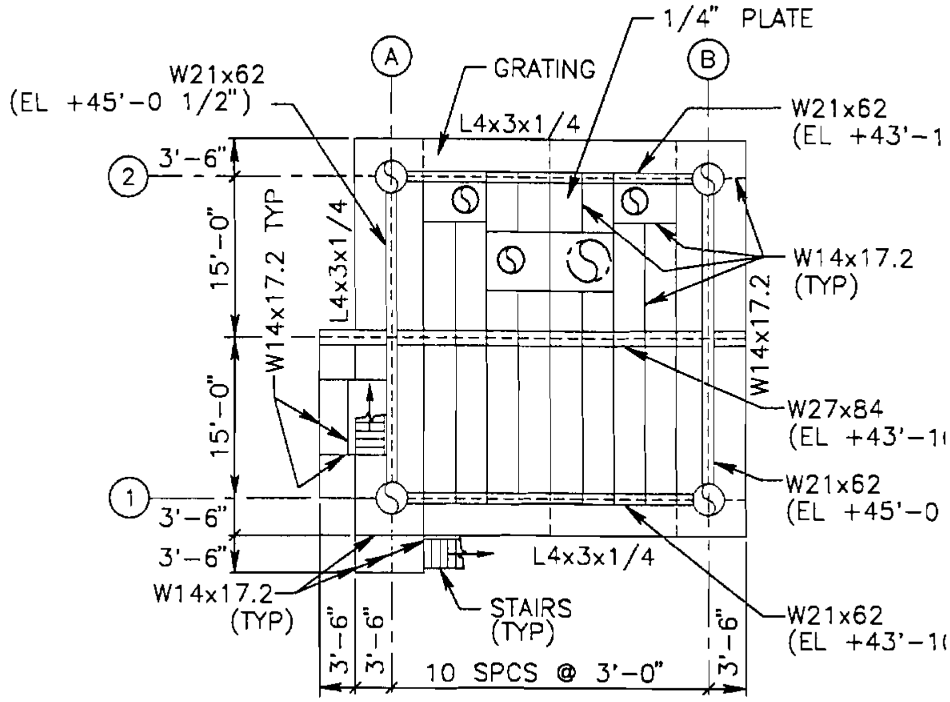
Undrained Shear Strength, S_u (k/ft^2)	Initial Static Stiffness Parameter, K_s (k/in^3)
1.0 - 2.0	0.5
2.0 - 4.0	1.0
4.0 - 8.0	0.2



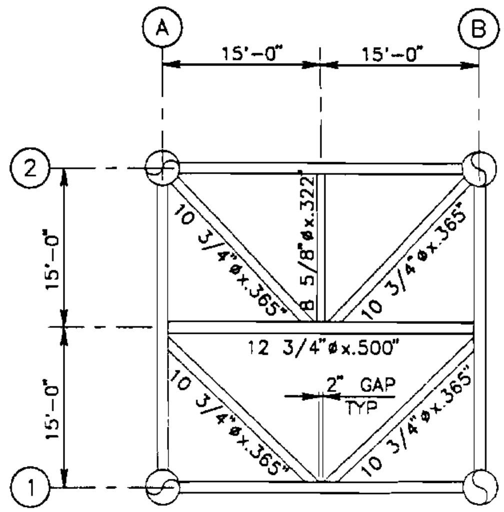
Note: Boat Landing and Barge Bumpers are located on Platform East and South

Figure 2-1: Key Plan

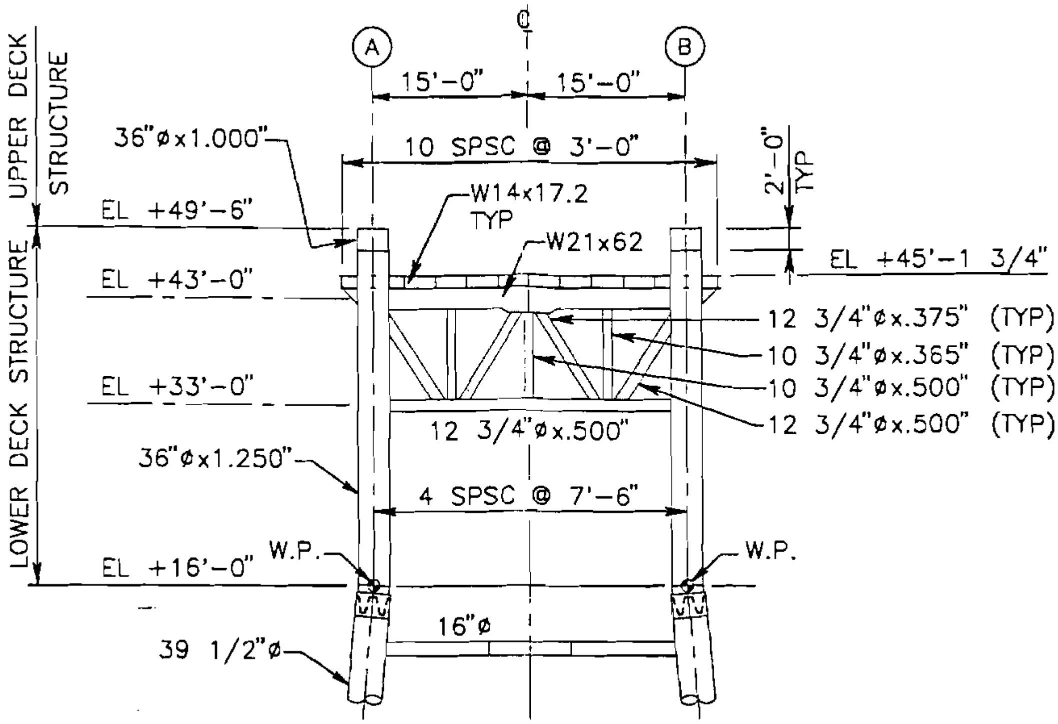




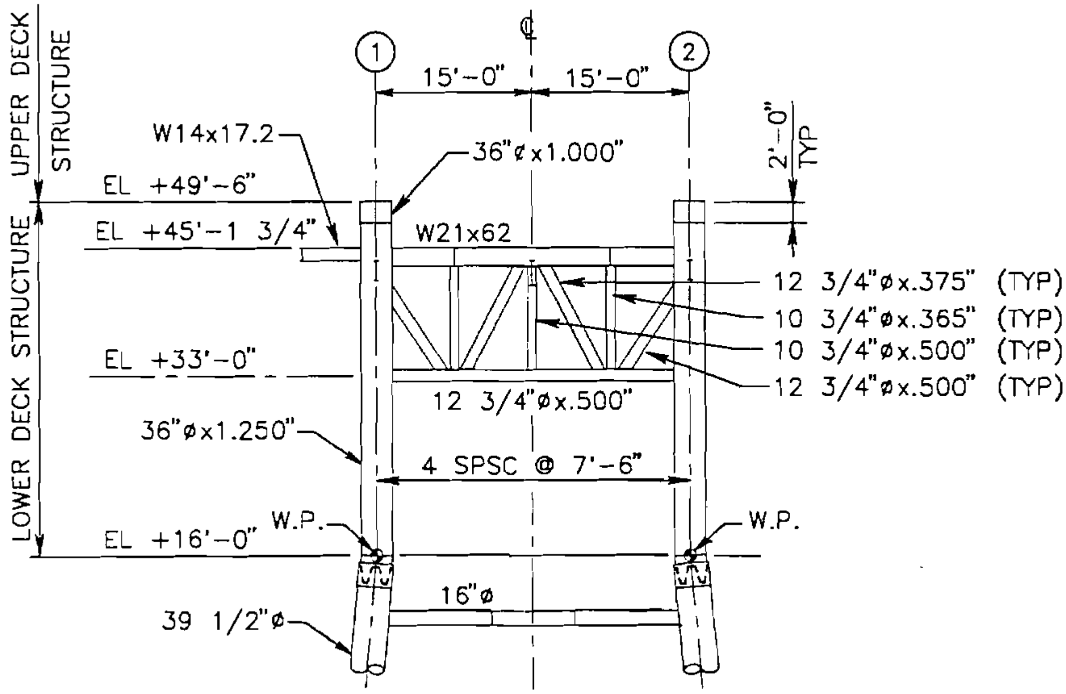
ELEV +45'-1 3/4"



ELEV +33'-0"

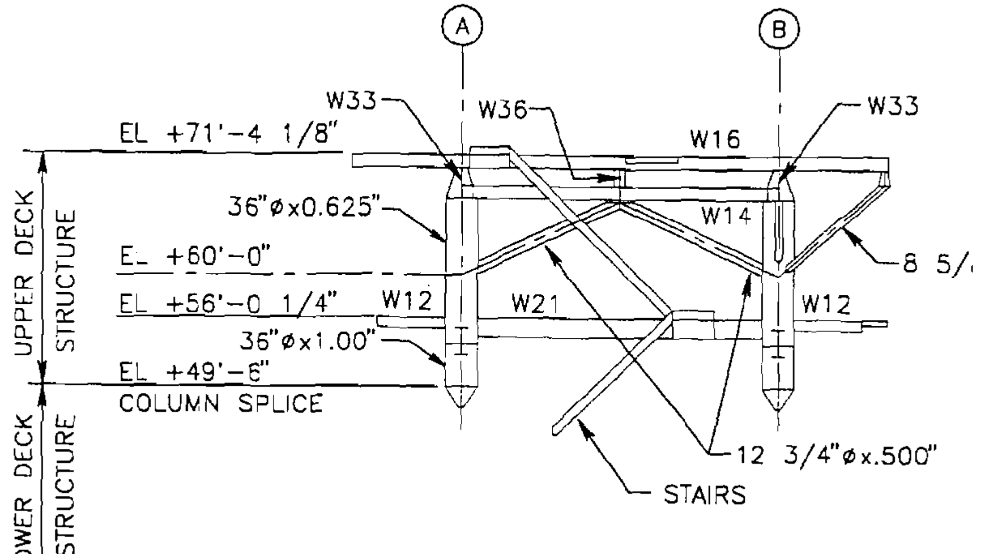


ROW 2

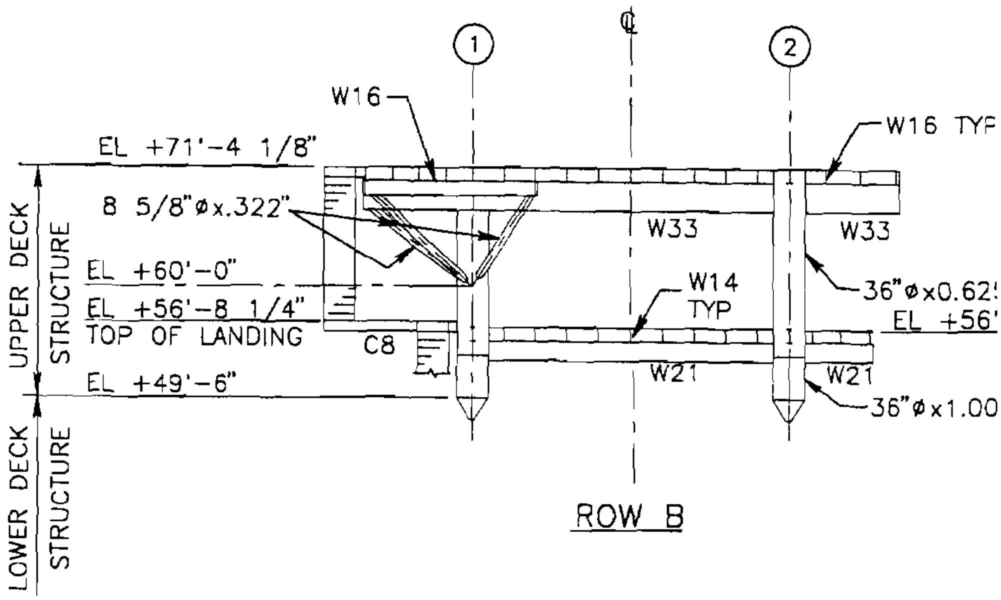


ROW A

Figure 2–2:
Lower Deck Structural Details

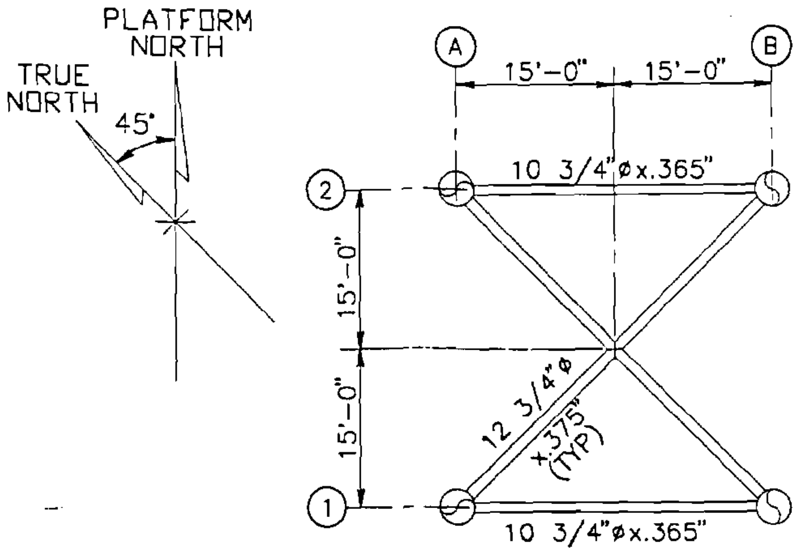

A
B
W33
W36
W33
W16
EL +71'-4 1/8"
36"ø x0.625"
W14
EL +60'-0"
8 5/
EL +56'-0 1/4"
W12
W21
W12
36"ø x1.00"
EL +49'-6"
COLUMN SPLICE
12 3/4"ø x.500"
STAIRS
UPPER DECK STRUCTURE
LOWER DECK STRUCTURE

ROW 1


1
2
W16
W16 TYP
EL +71'-4 1/8"
8 5/8"ø x.322"
W33
W33
36"ø x0.62
EL +60'-0"
W14
TYP
EL +56'-8 1/4"
TOP OF LANDING
EL +56'
C8
W21
W21
EL +49'-6"
36"ø x1.00
UPPER DECK STRUCTURE
LOWER DECK STRUCTURE

ROW B

"øx.322"



ELEV +64'–4 3/4"

Note: Jacket column spacing of 29'–11 5/8" (from field measurements) at El +49'–6" instead of 30'–0" in design drawings.

"
-0 1/4"

Figure 2–3:
Upper Deck Structural Details
Elevations and Plan at El +64'–4 3/4"

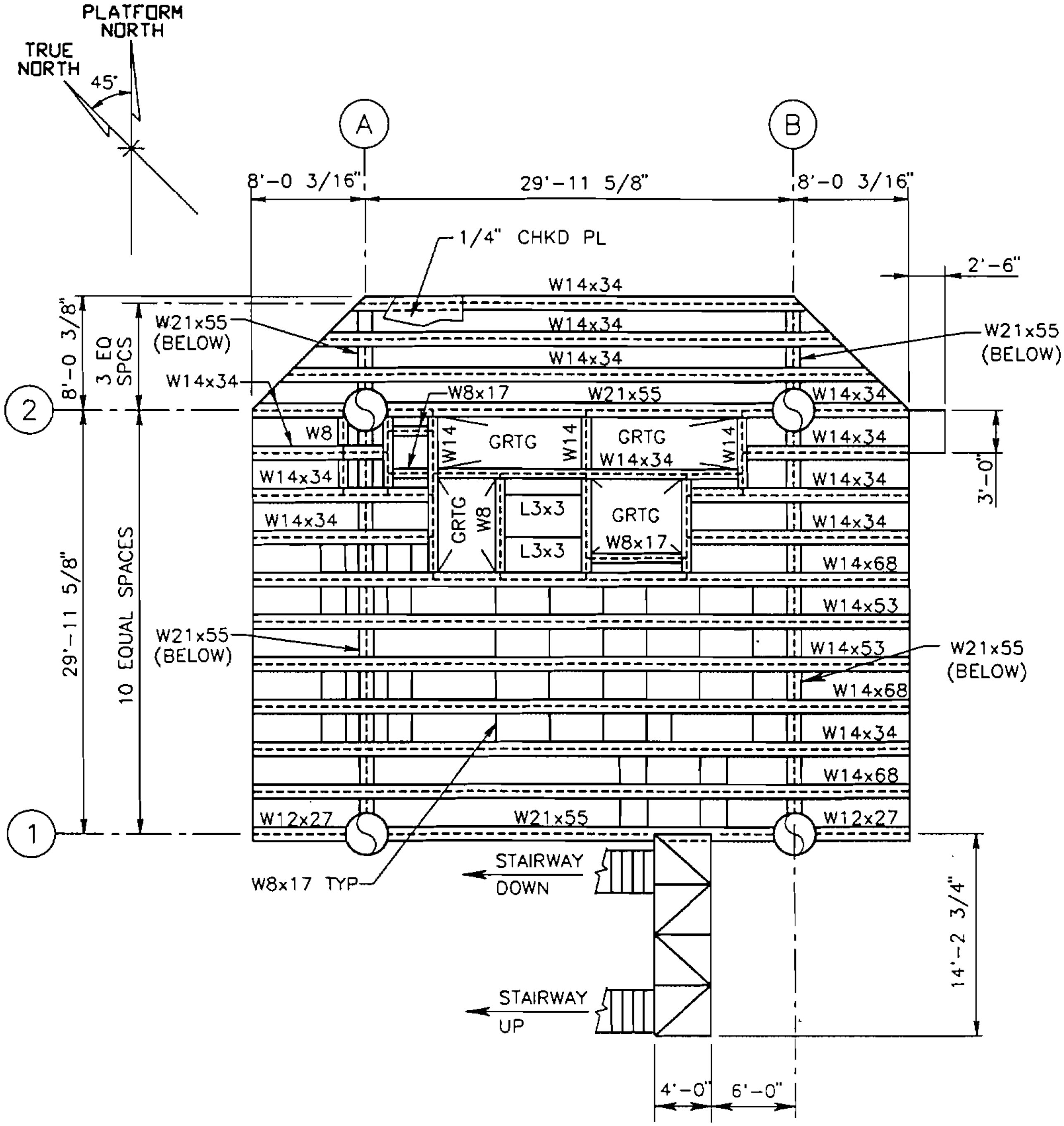


Figure 2-4:
Upper Deck Structural Details
Framing Plan at El +56'-0 1/2"

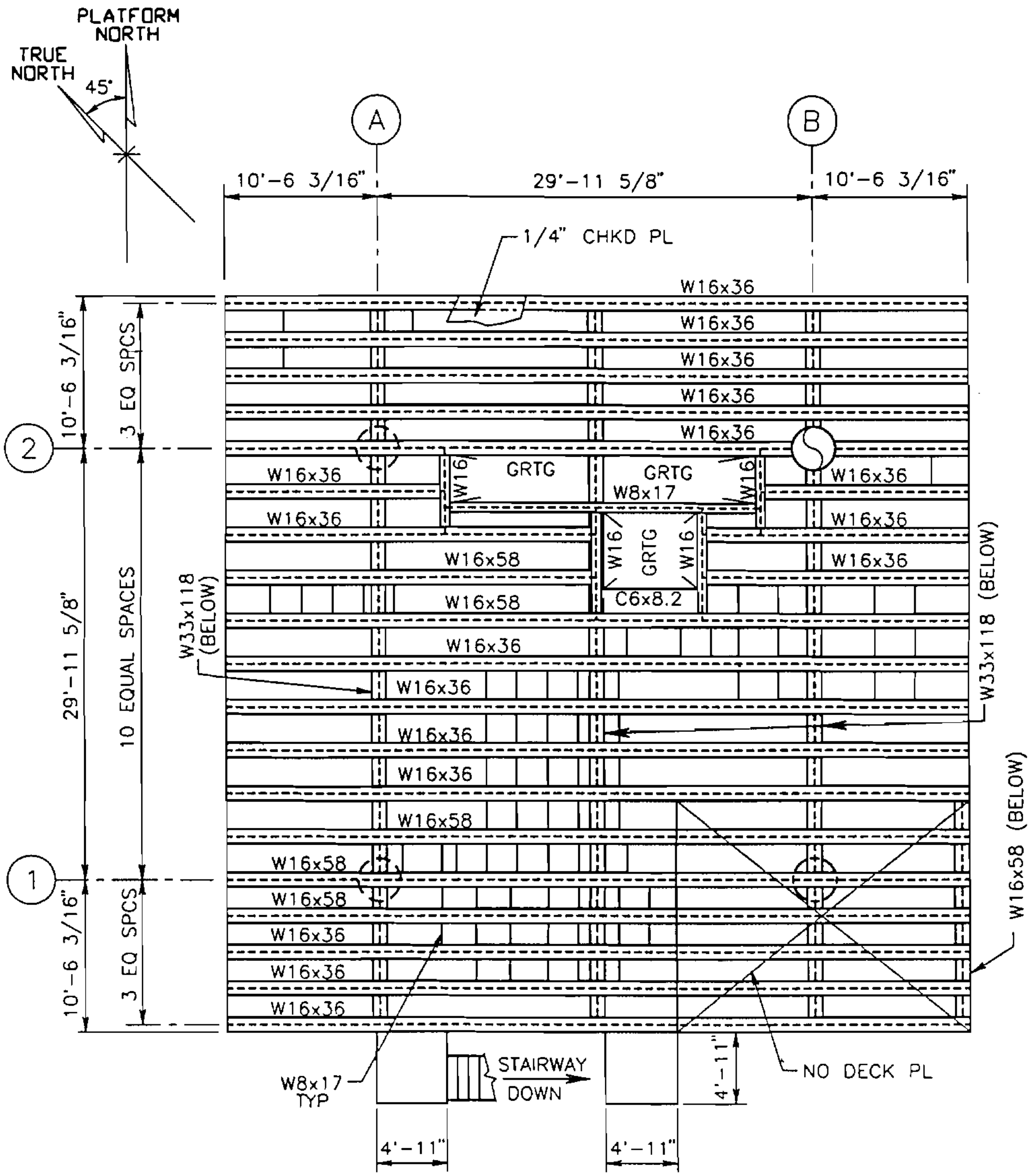
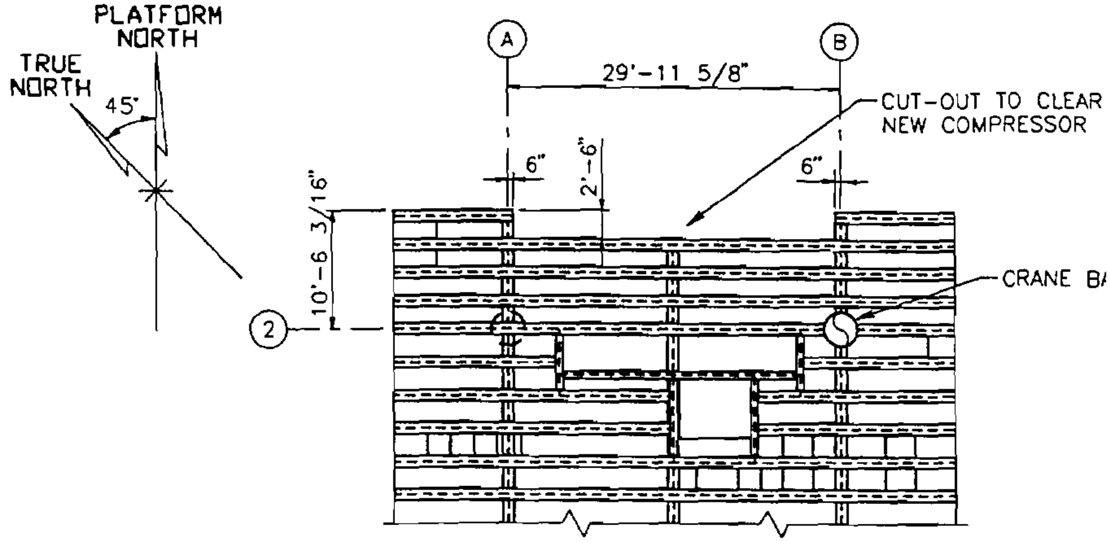
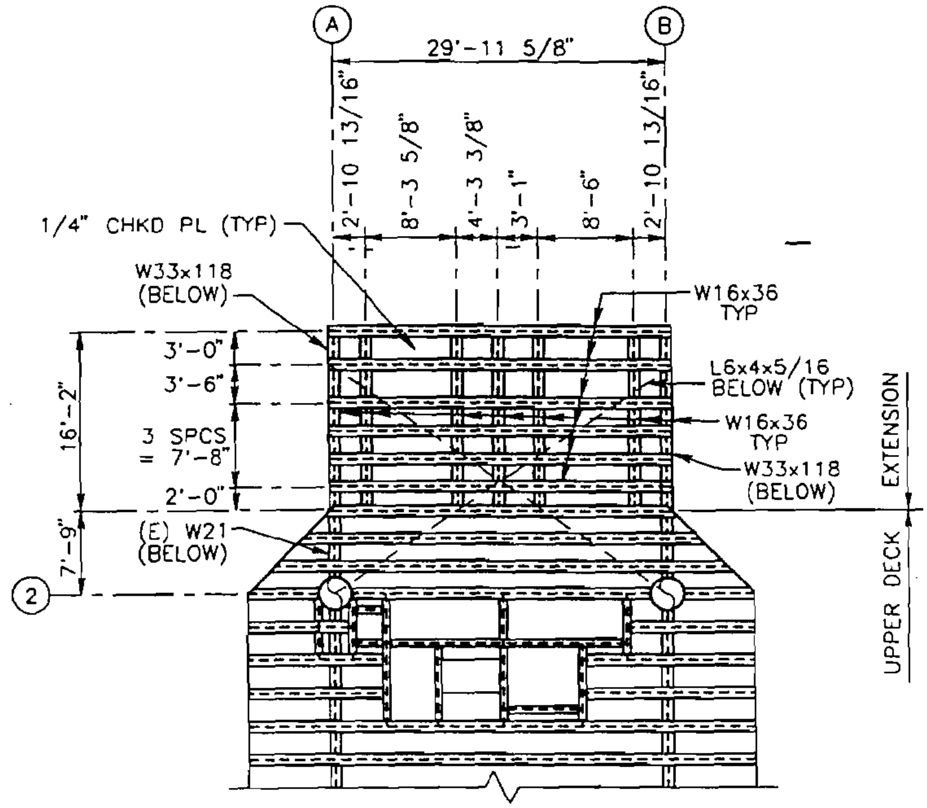


Figure 2–5:
Upper Deck Structural Details
Framing Plan at El +71'-4 1/8"



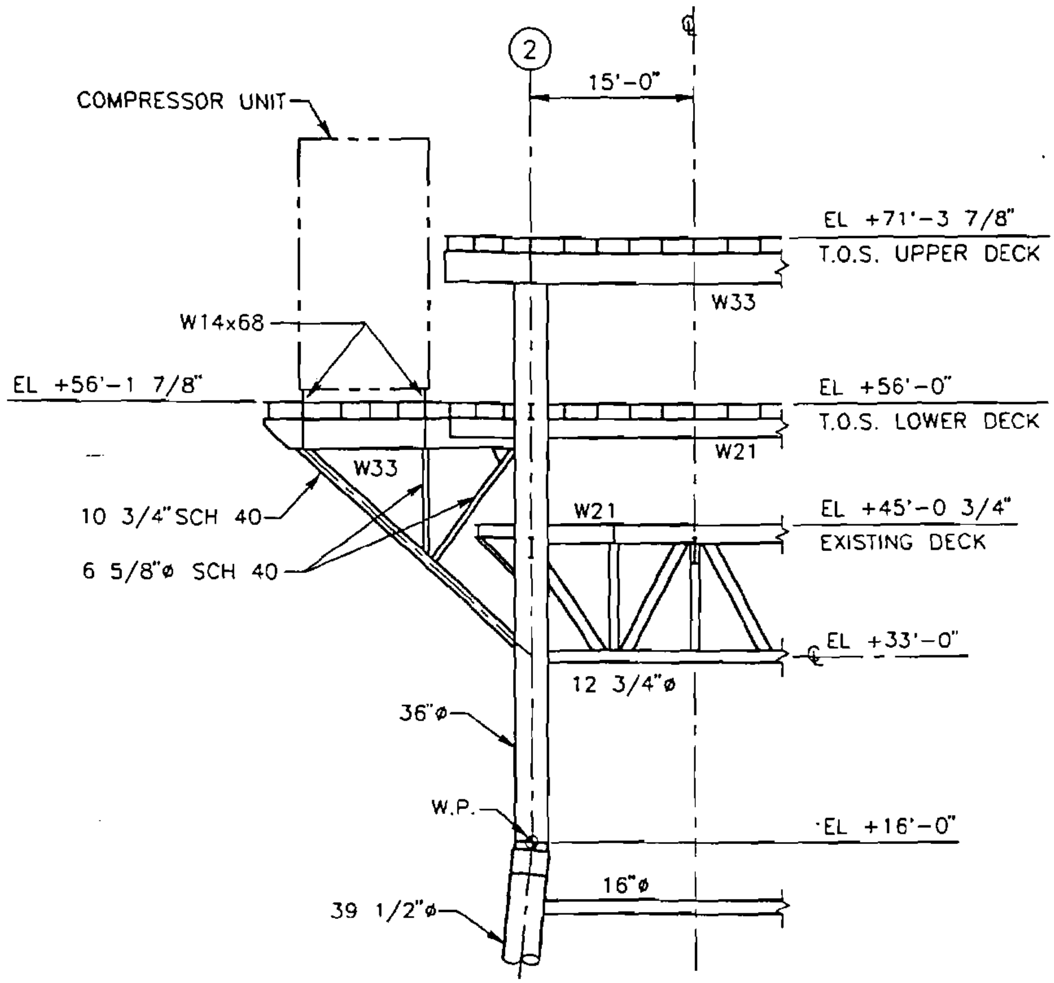
CUT-OUT AT EL +71'-4 1/8"



EXTENSION AT EL +56'-0 1/4"

INIT

SE



ELEVATION AT ROW A
ROW B SIMILAR

Figure 2-6:
Upper Deck Structure
Modifications for New Gas Generator

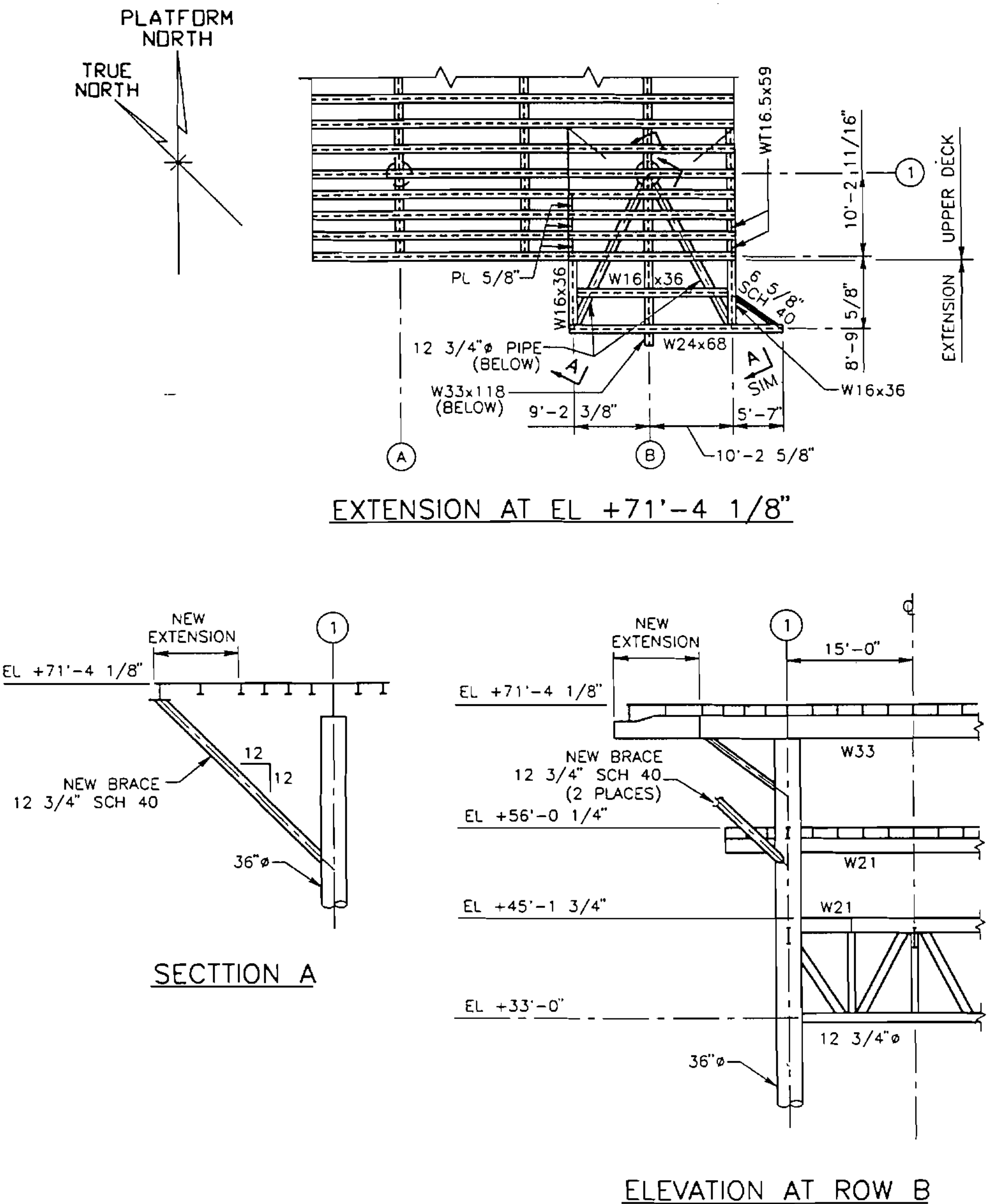
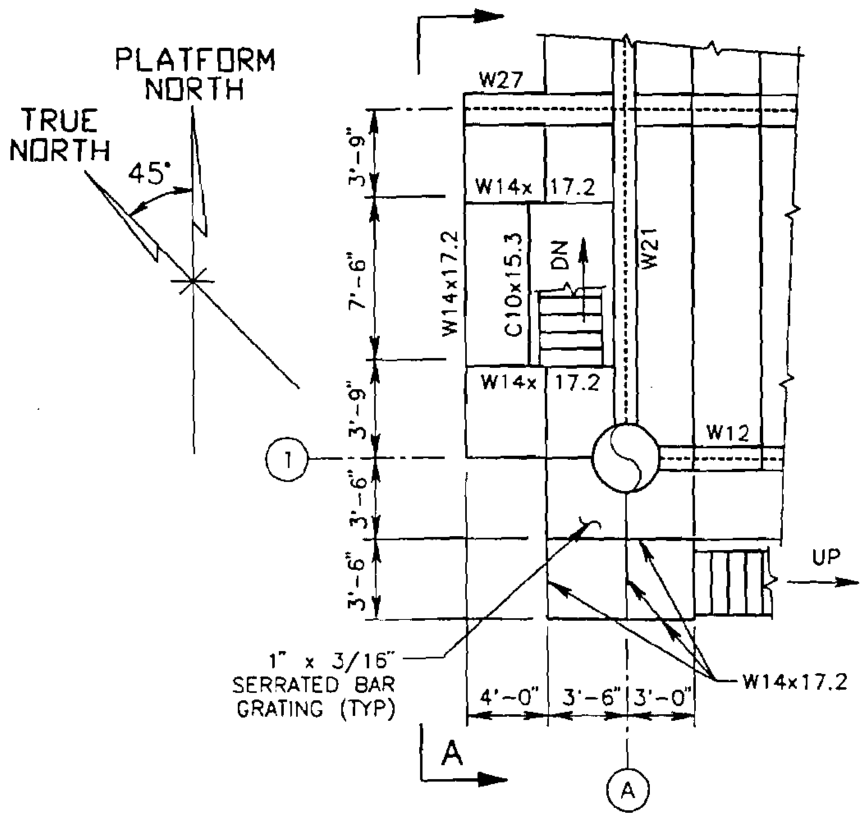
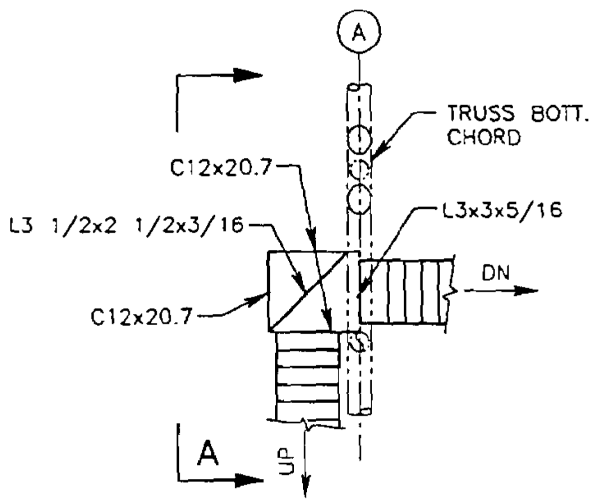


Figure 2-7:
Upper Deck Structure
Extension for New Quarters



PLAN AT EL +45'-1 3/4"

T.O.S



PLAN AT EL +33'-6 5/8"

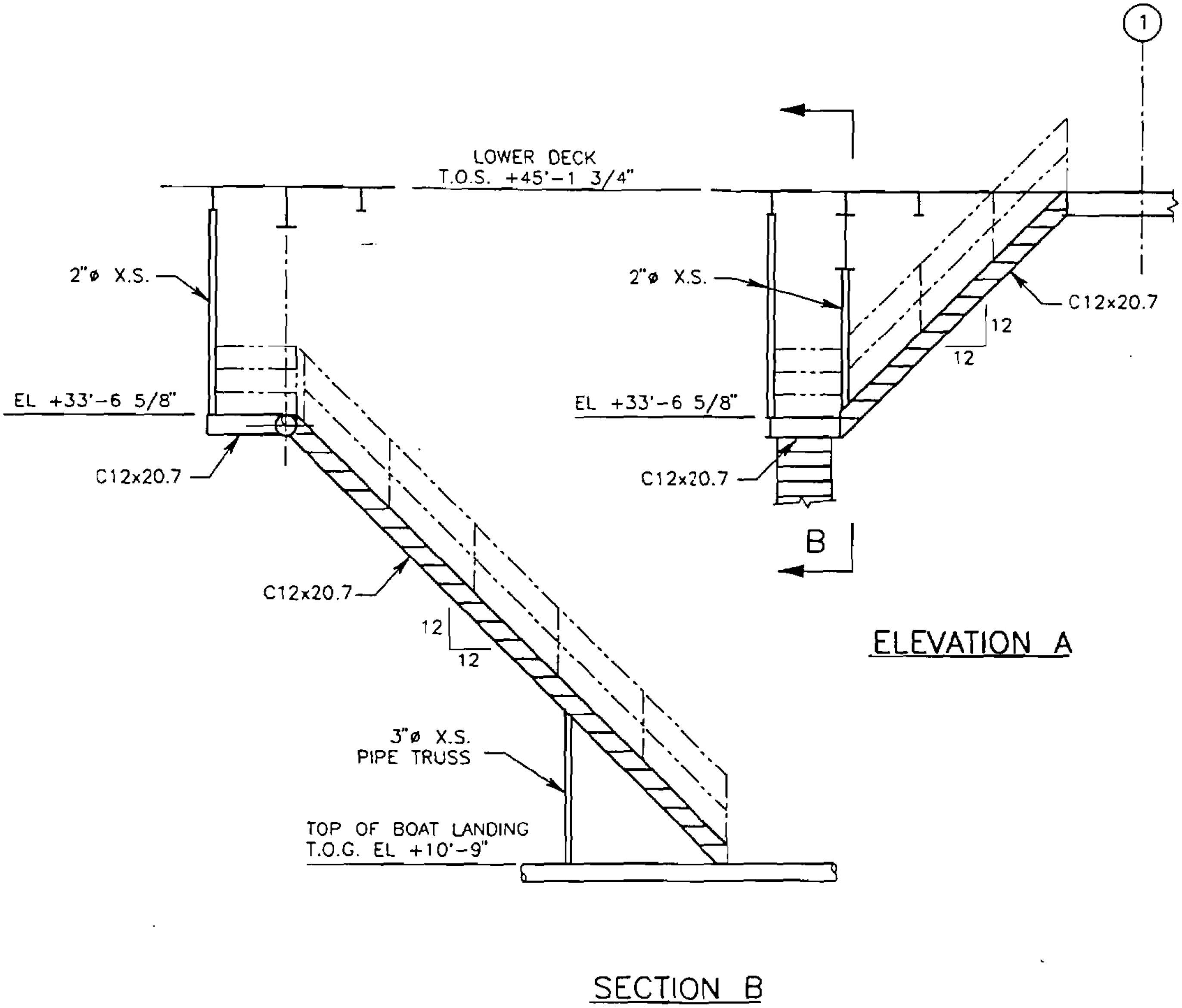
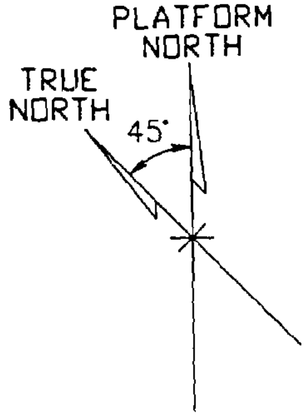
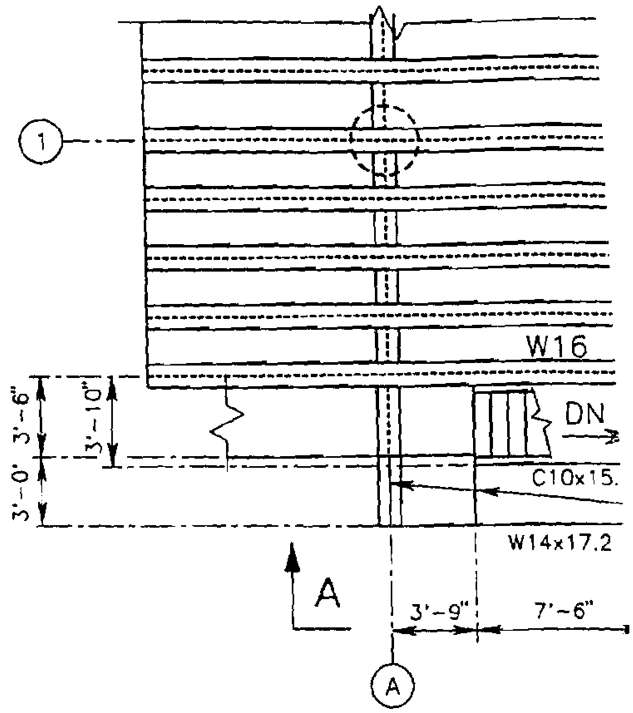
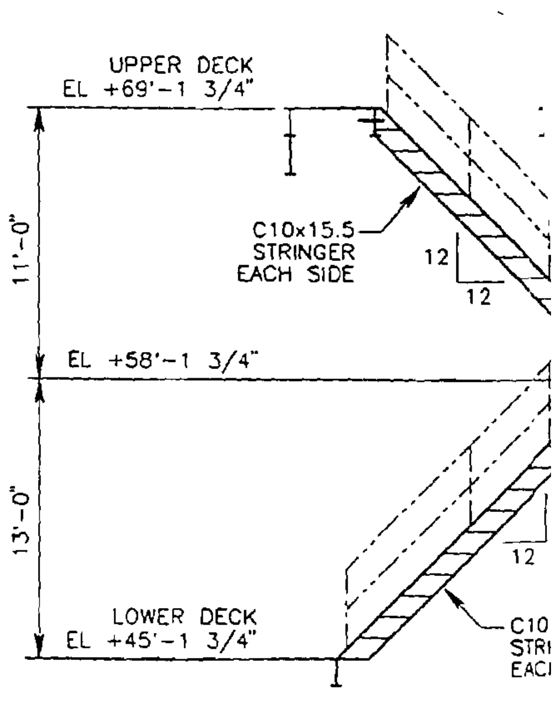


Figure 2-9:
Stair Details – Lower Deck to Boat Landing

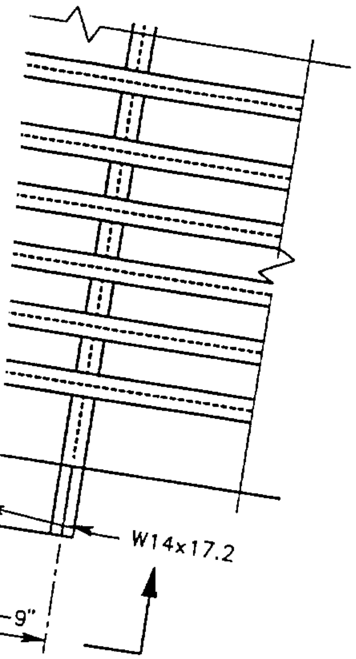




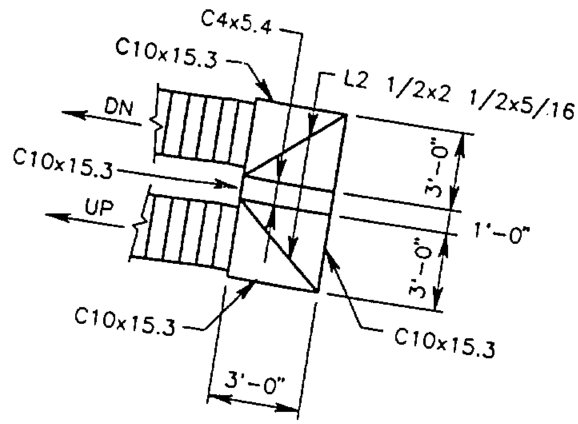
PLAN AT EL +69'



SECTION A



3/4"



PLAN AT EL +58'-1 3/4"

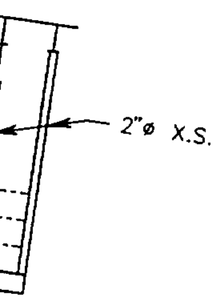
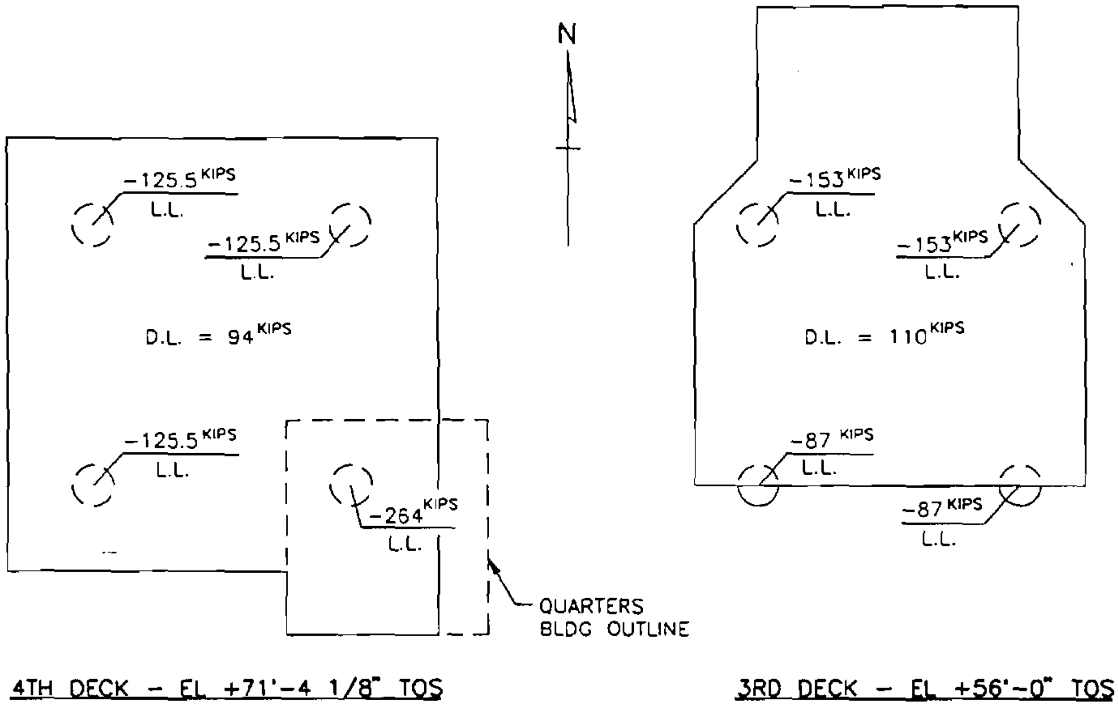
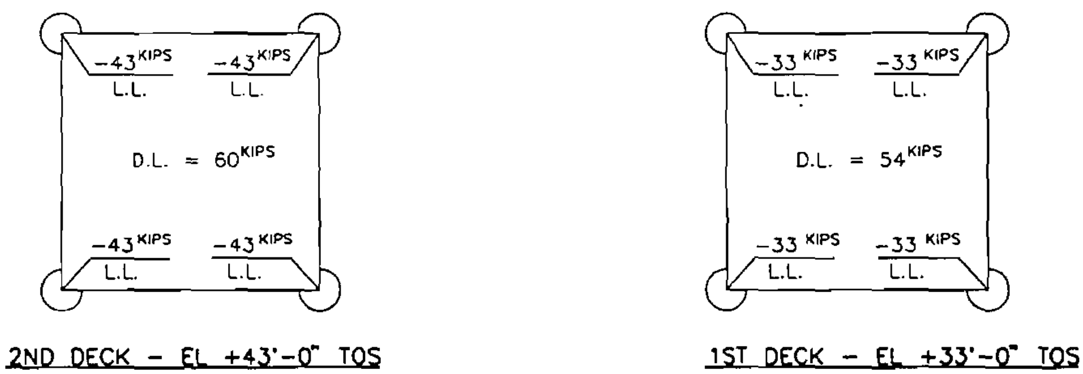


Figure 2-8:
Stair Details –



UPPER DECK PLANS

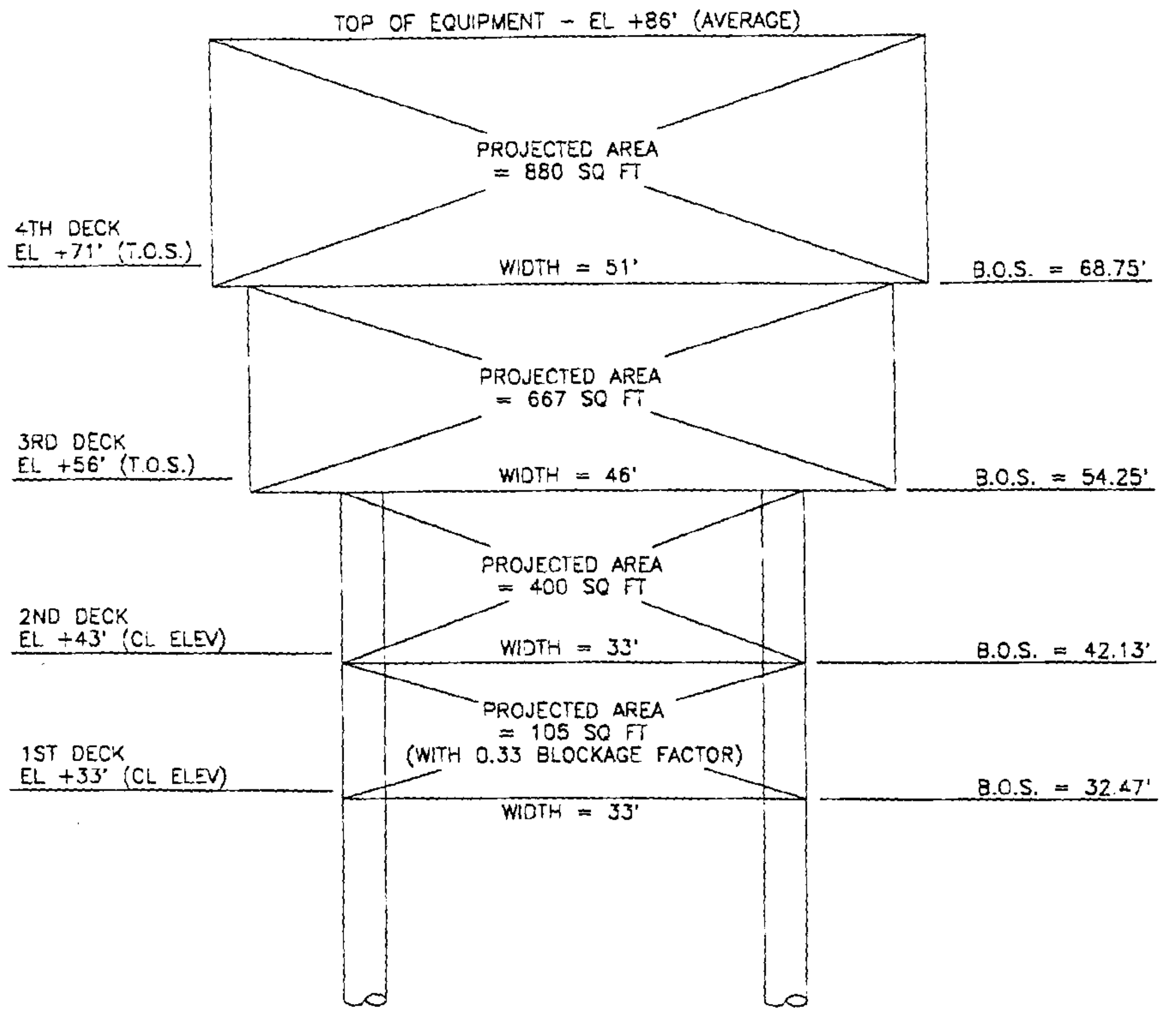


LOWER DECK PLANS

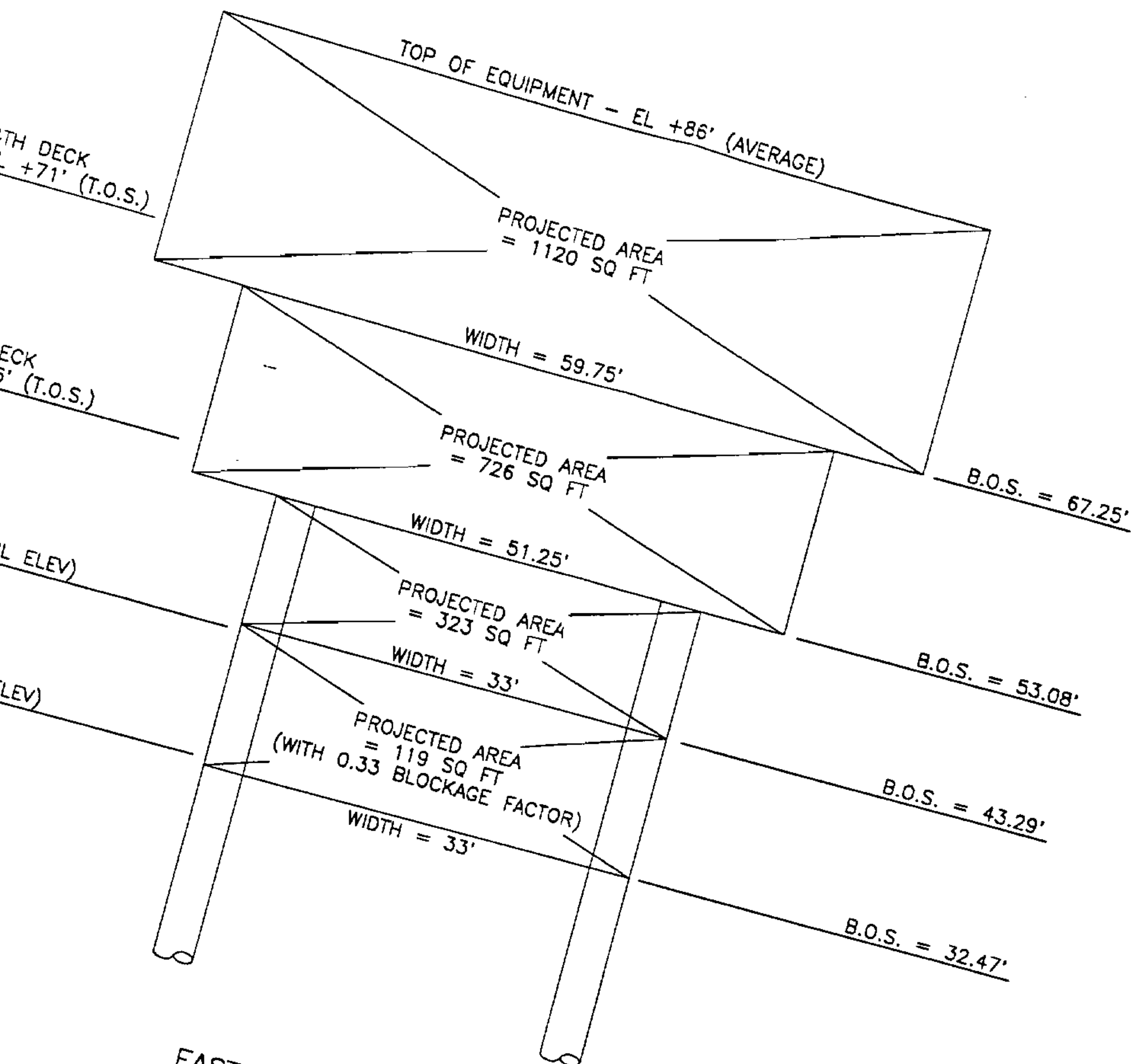
DECK	EQUIPMENT
1ST DECK	HORIZONTAL RUNS OF RISERS AND 4' x 8' x 4' SUMP TANK
2ND DECK	GENERATOR SET, HEADER, WELL HEADS (CONGESTED)
3RD DECK	(CONGESTED)
4TH DECK	EQUIPMENT WITH QUARTERS (PLAN 24' x 25'; HEIGHT 24') ON SOUTHEAST CORNER

NOTES:
1. L.L. = LIVE LOAD D.L. = DEAD LOAD
2. DEAD LOAD OF DECK COLUMNS (EL +16' TO +33') = 32 KIPS (I.E., 8 KIPS/LEG)

Figure 2-10: Deck Loading



NORTH/SOUTH PROJECTIONS



EAST/WEST PROJECTIONS

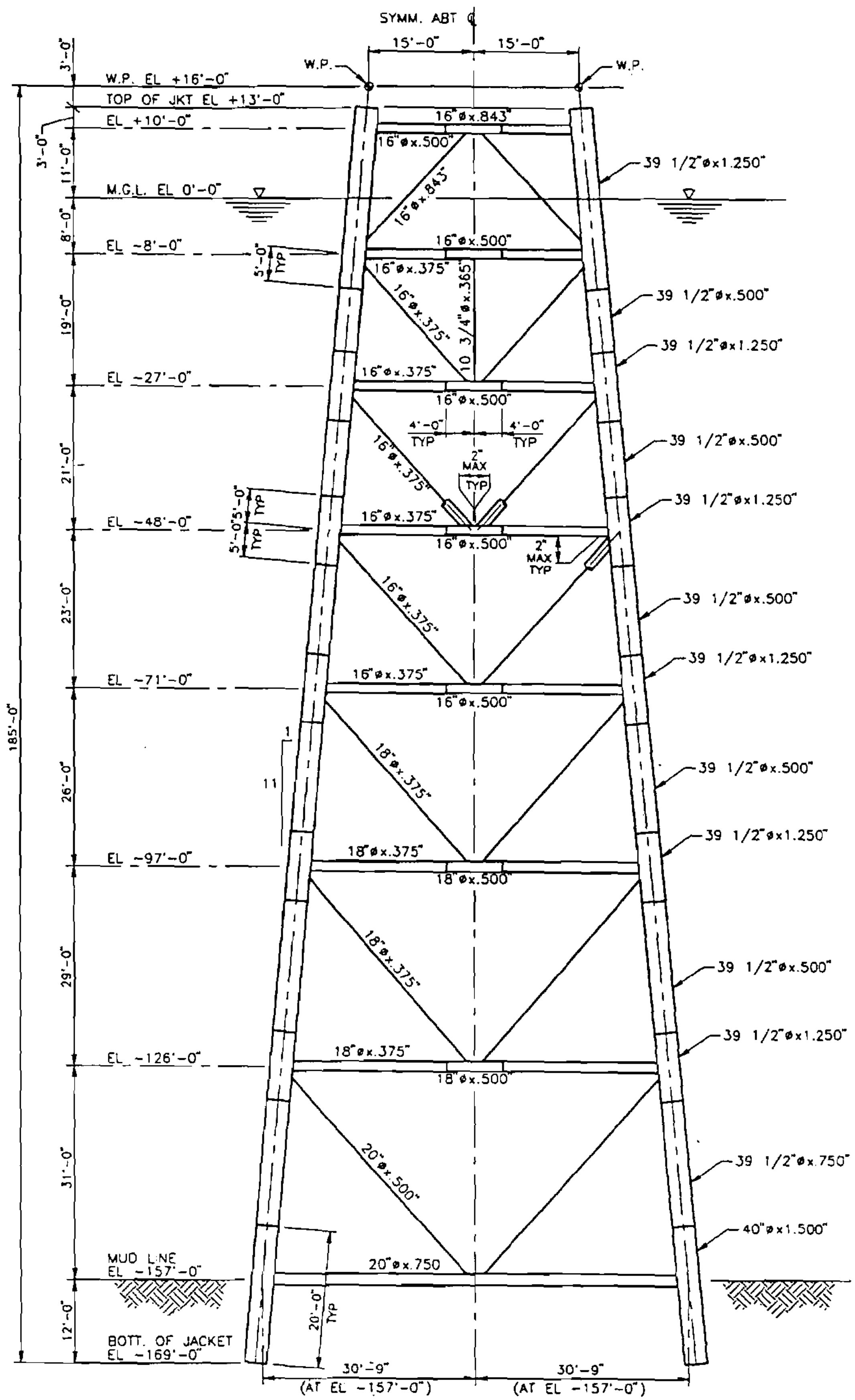
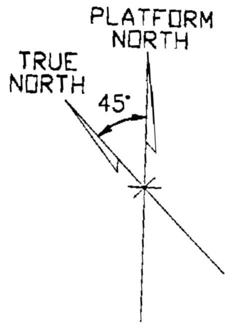
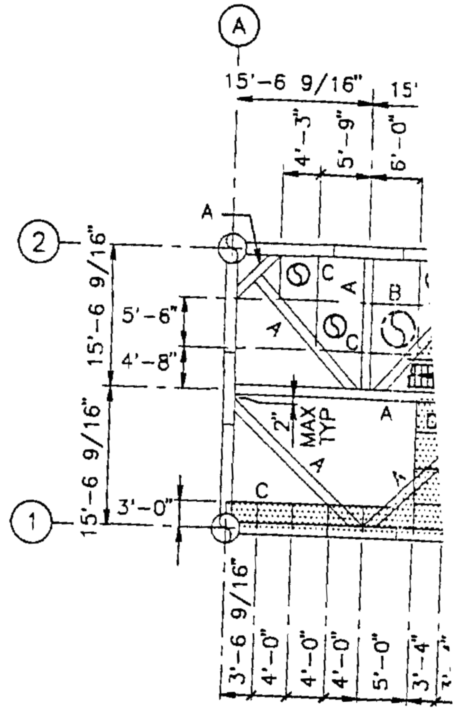


Figure 2-12: Jacket - Typical Elevation



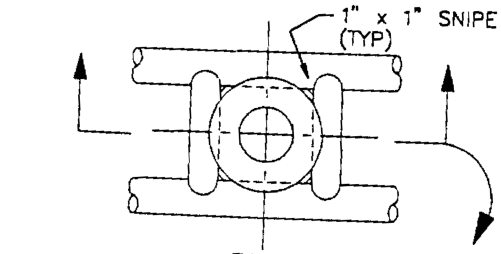
MEMBER	SIZE
A	12 3/4"x.375"
B	8 5/8"øx.322"
C	6 5/8"øx.280"
D	2 1/2"øx.218"



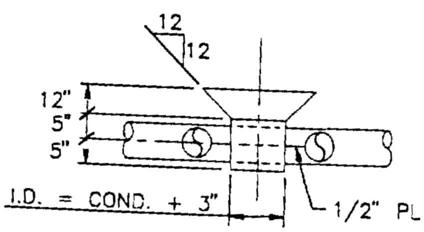
ELEV +10'-0"

T.O.S. EL +10'-8'

NOTE: BOAT LANDINGS & I
ON SOUTH & EAST SI
SEE FIGURE 2-15 FOR D

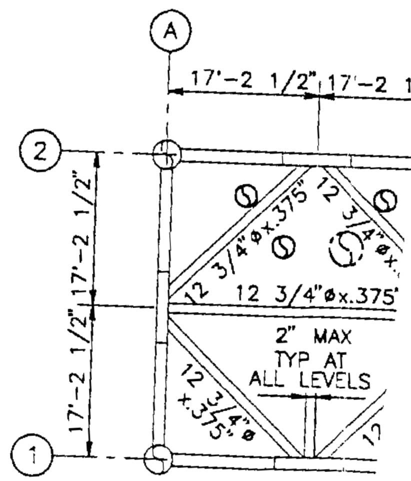


PLAN



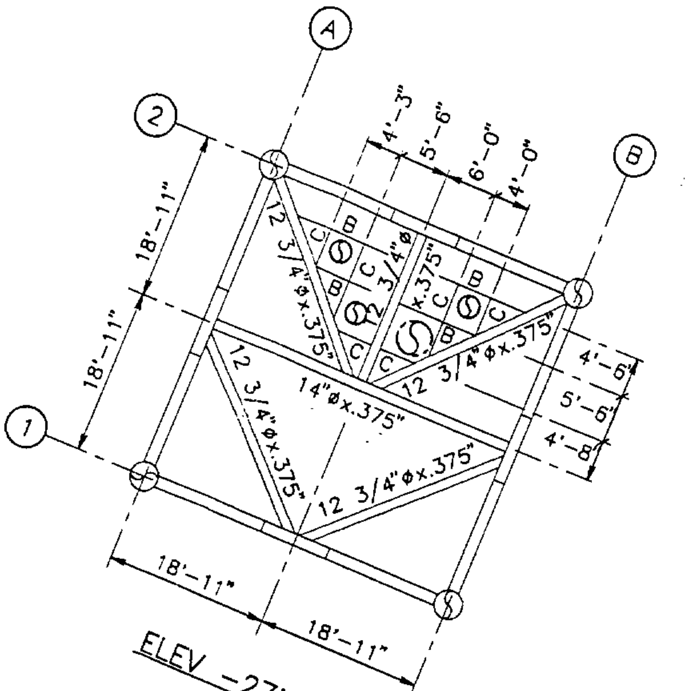
SECTION

CONDUCTOR GIUDE

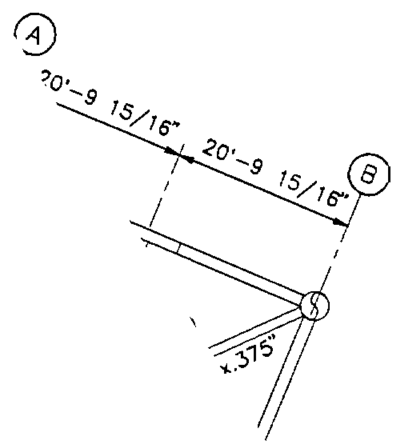


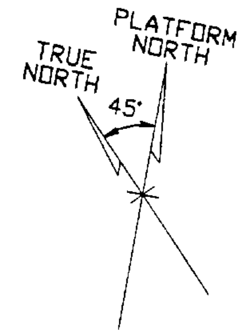
ELEV -8'-

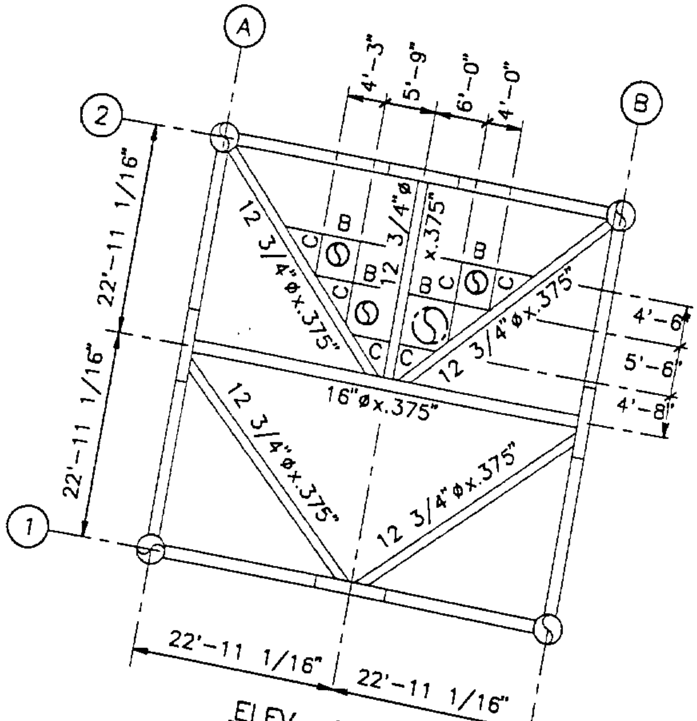

B
-6 9/16"
A
1" x 3/16"
SERRATED BAR
GRATING (TYP)
5 SPACES
AT 4'-0"
3'-6 9/16"
MPERS
ES
TAILS
B
2"
375"
3/4"ø
x.375"
0"


A
2
B
1
4'-3"
5'-6"
6'-0"
4'-0"
18'-11"
18'-11"
12 3/4"ø x.375"
12 3/4"ø x.375"
12 3/4"ø x.375"
12 3/4"ø x.375"
12 3/4"ø x.375"
12 3/4"ø x.375"
14"ø x.375"
B
C
4'-6"
5'-6"
4'-8"
18'-11"
18'-11"

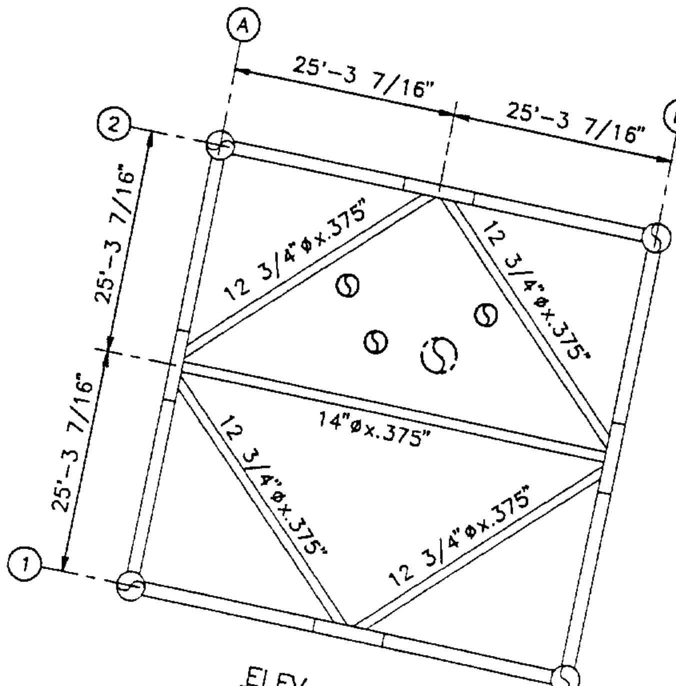
ELEV -27'-0"


A
20'-9 15/16"
20'-9 15/16"
B
x.375"



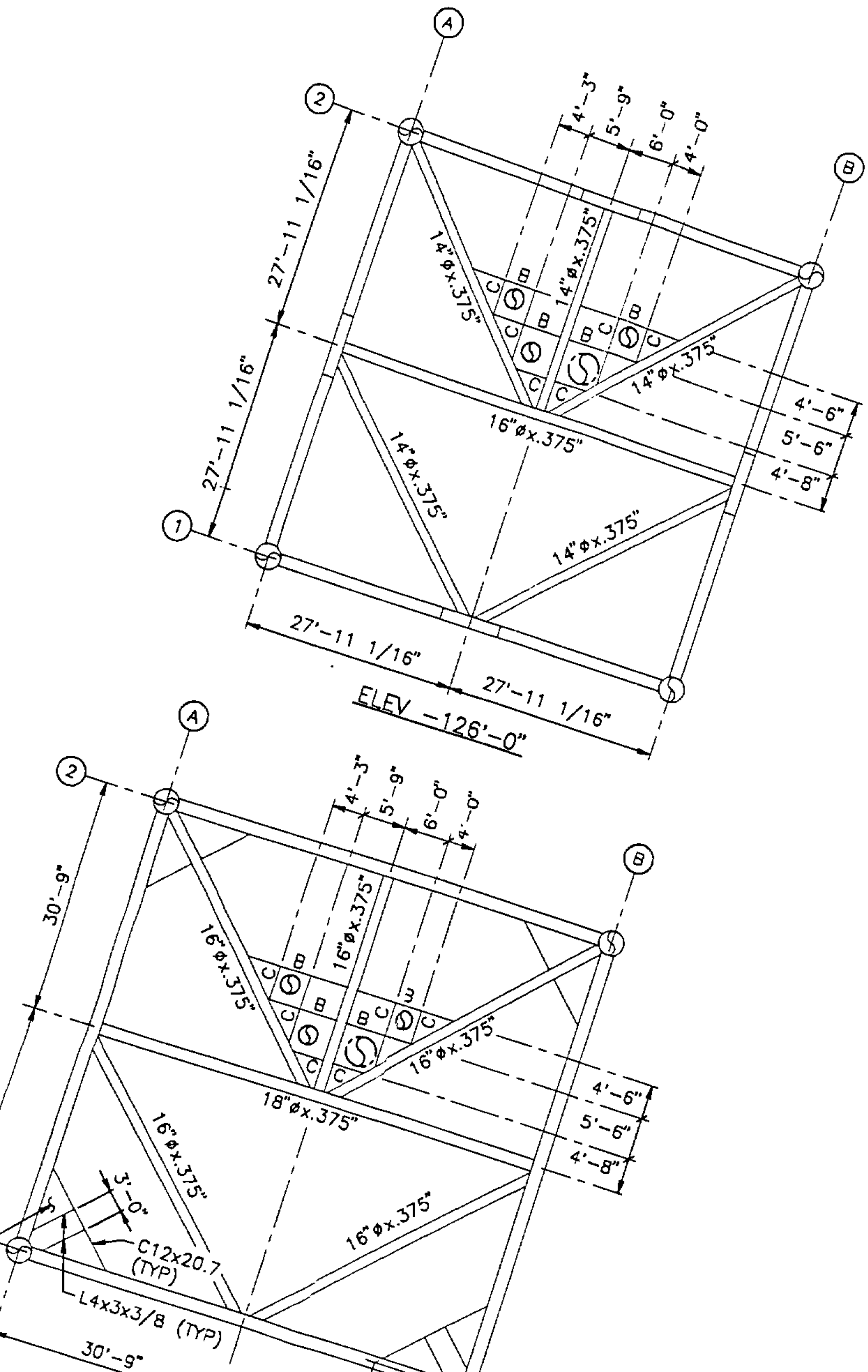


ELEV -71'-0"

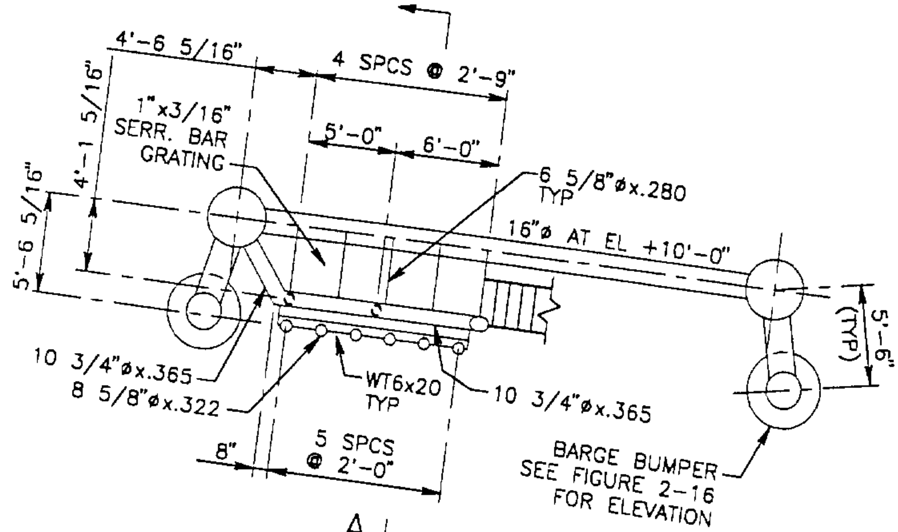


ELEV -97'-0"

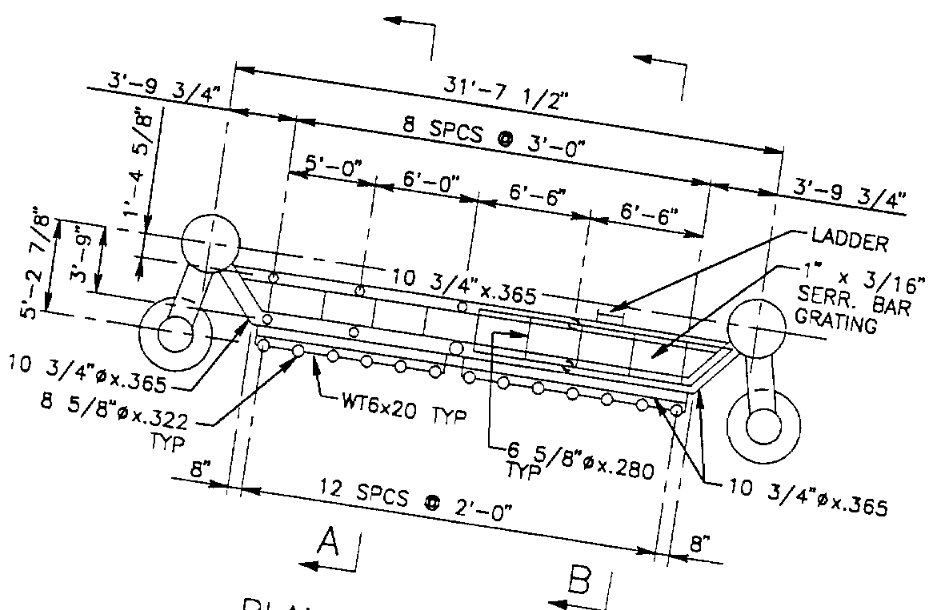
MEBER	
	8
	6



A
B
1
2
27'-11 1/16"
4'-3"
5'-9"
6'-0"
4'-0"
14"øx.375"
16"øx.375"
4'-6"
5'-6"
4'-8"
ELEV -126'-0"
30'-9"
16"øx.375"
18"øx.375"
3'-0"
C12x20.7
(TYP)
L4x3x3/8 (TYP)

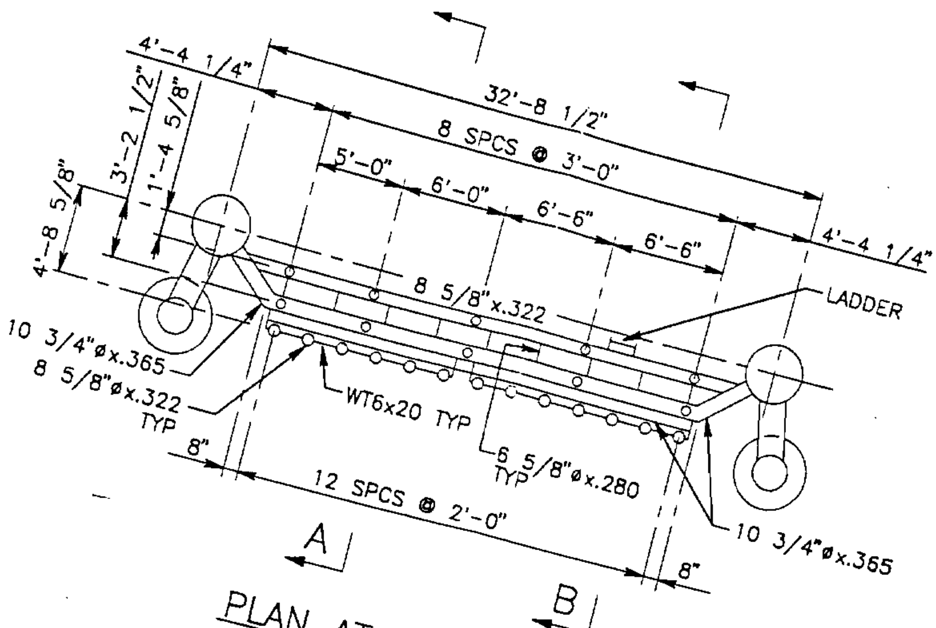


PLAN AT EL +10'-2 5/8"

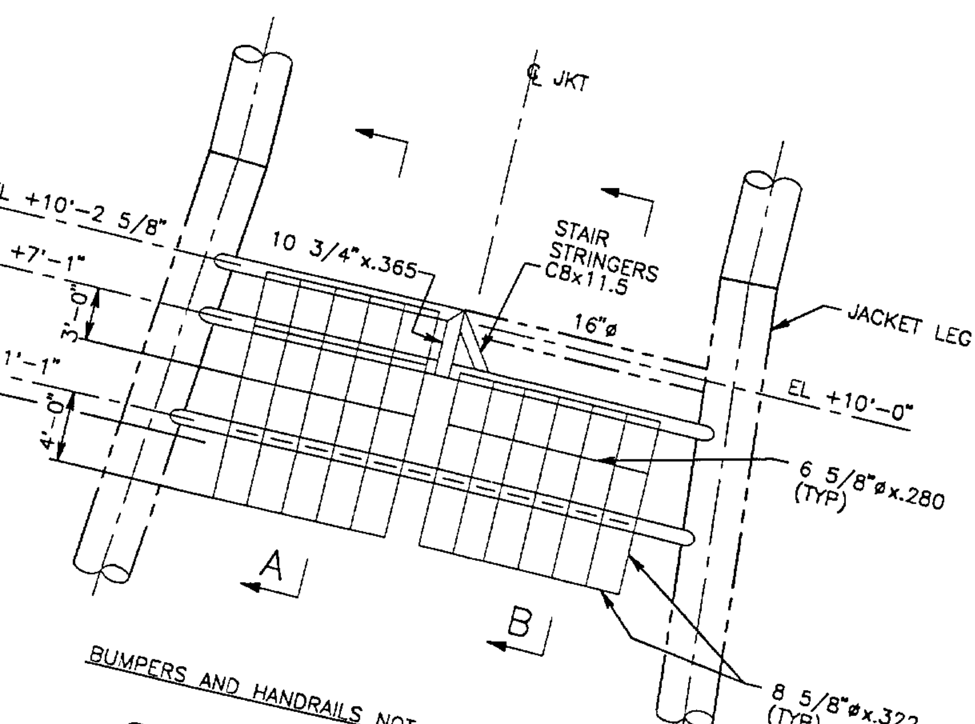


PLAN AT EL +7'-1"

M.G.L.
EL 0'-0

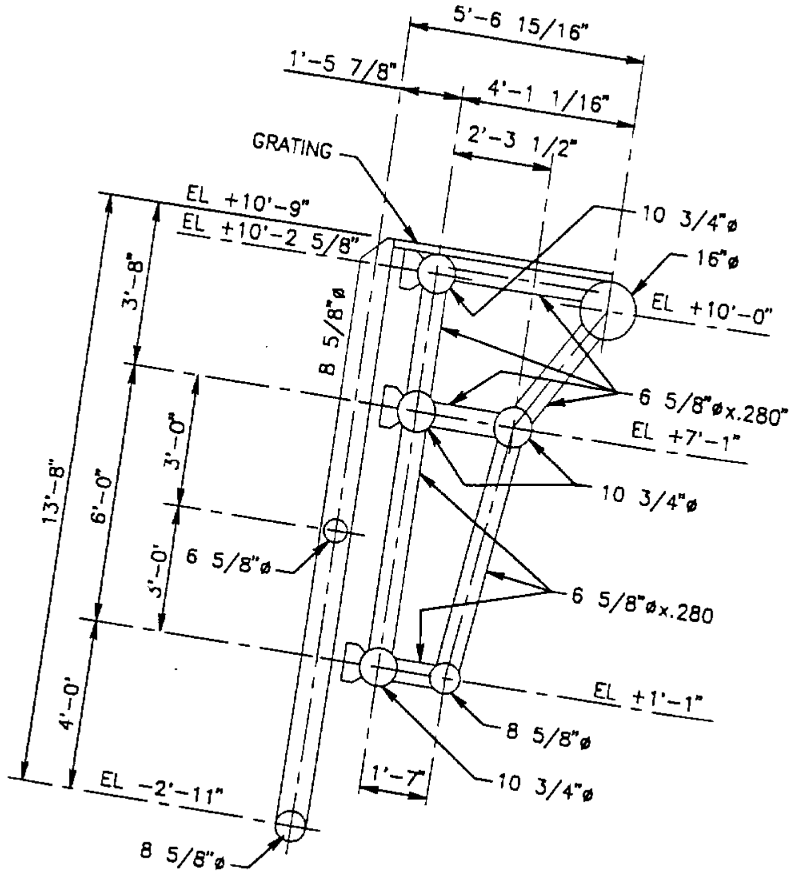


PLAN AT EL +1'-1"

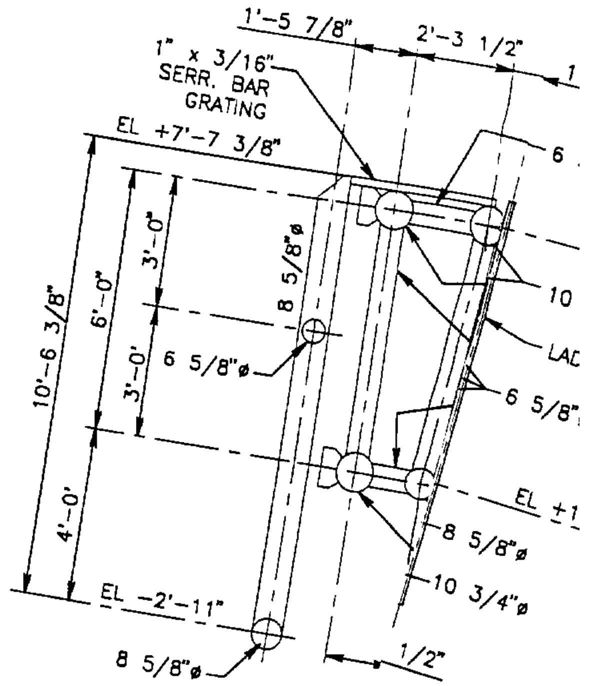


BUMPERS AND HANDRAILS NOT SHOWN FOR CLARITY

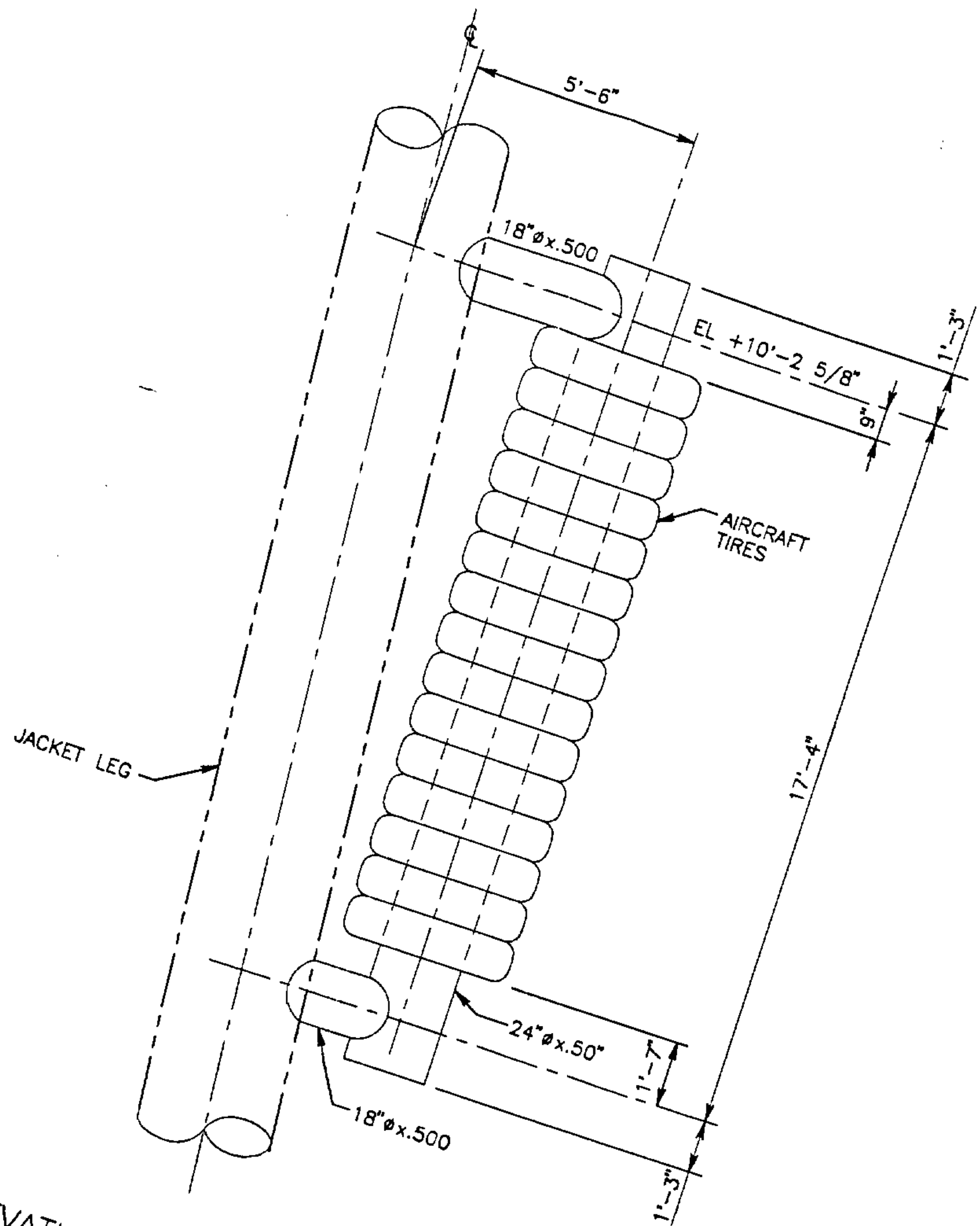
SOUTH ELEVATION SHOWN
EAST ELEVATION SI...



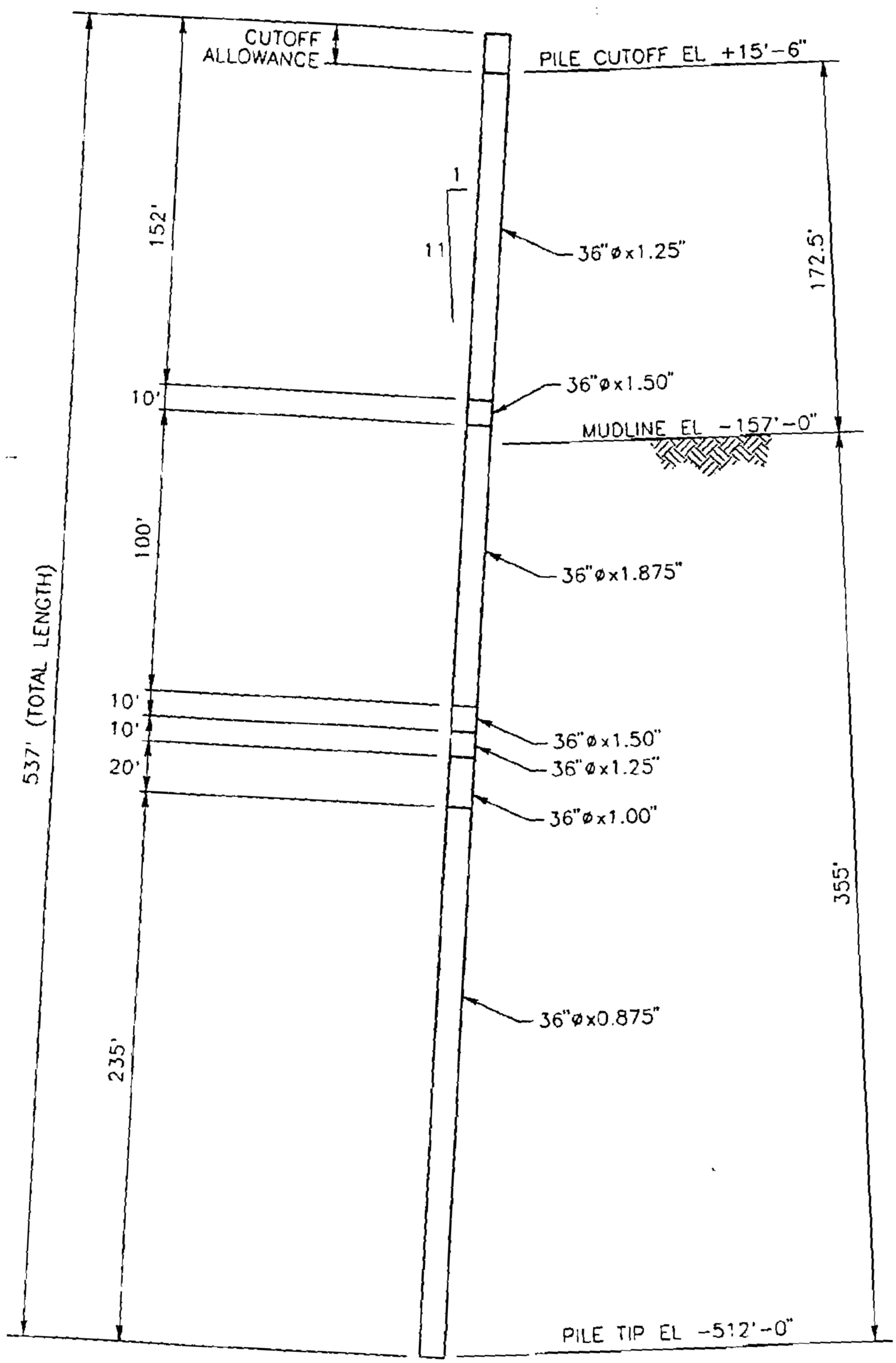
SECTION A
FROM FIGURE 2-15



SECTION
FROM FIGURE



ELEVATION — BARGE BUMPER
(4 REQUIRED TOTAL)



NOTE: FIELD SPLICES NOT SHOWN

Figure 2-17: Pile Detail

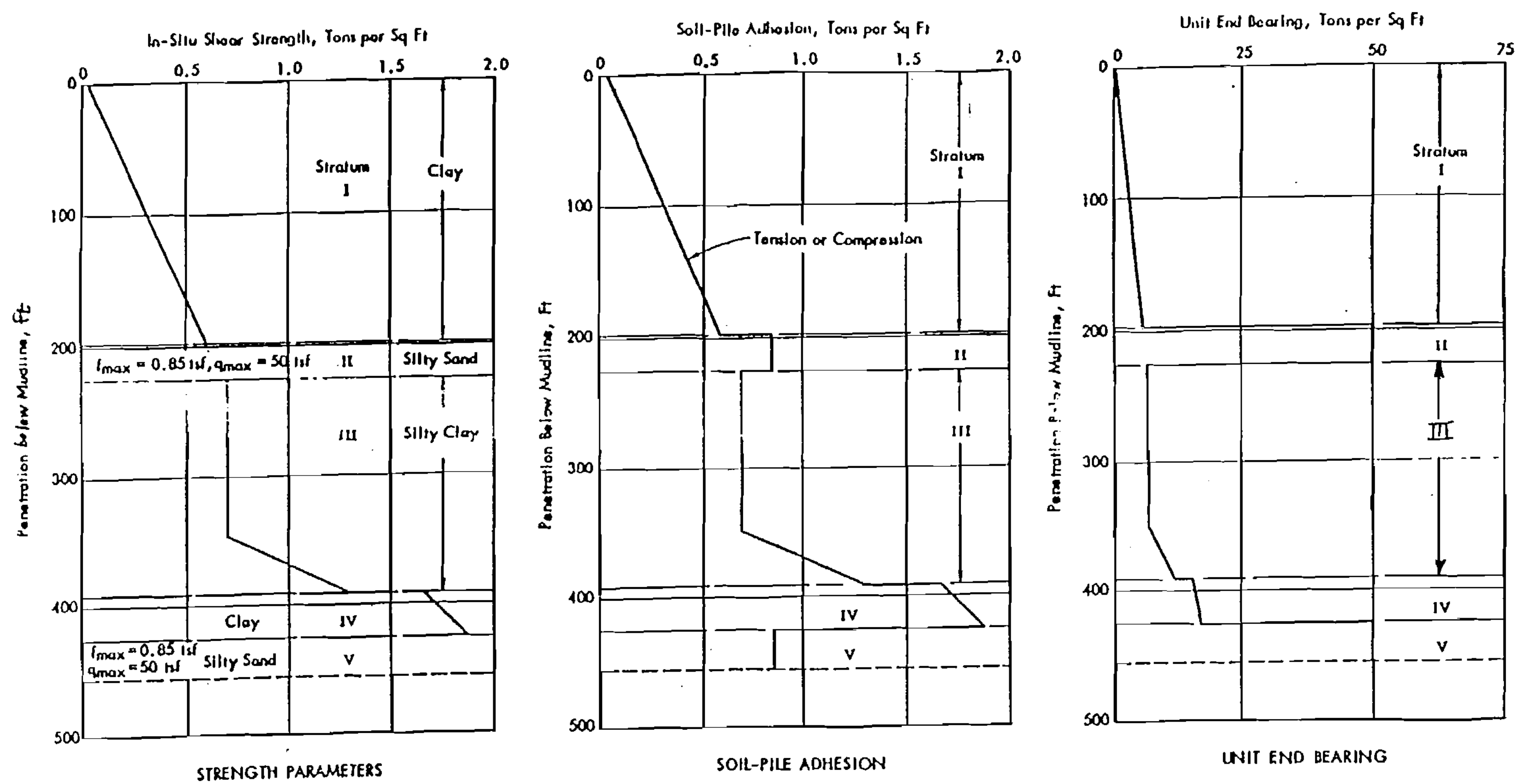


Figure 2-18 Soil Strength Parameters

The metocean criteria required for computation of the metocean loads (platform base shear and overturning moment) for the benchmark platform are defined in this section. The criteria is identified for the following three cases:

- Guideline API RP 2A, 20th edition design
- Section 17 design level significant-environmental impact
- Section 17 ultimate strength significant-environmental impact

The metocean criteria are identified for eight principal directions shown in Figure 3-1, to provide an example application of the guidelines. However, for analysis of the benchmark platform, the majority of JIP participants selected only three principal directions (Directions 1, 2, and 3) which define the maximum loading (see Section 5 for further details of the JIP results).

3.1 API RP2A-WSD 20th Edition Metocean Criteria

The platform is located in a region shown in Figure 3-2 [3], to which the 20th edition metocean criteria is applicable. The water depth is assumed to be equal to Mean-Lower-Low-Water (MLLW).

Wave Heights: The guideline omnidirectional wave height per Figure 3-3 [3] is 63 feet. Wave heights, as a function of the required wave direction (for load computation), were obtained by using the guideline design factors per Figure 3-4 [3], as given in column 3 of the following table, and taking into account that these factors apply to the guideline direction of ±22.5 degrees. Interpolation should not be used.

Direction Number	Wave Direction Toward, Clockwise from North (degree)	Factors to apply to the Omnidirectional Wave Heights	Directional Wave Height (ft)
1	225	0.90	56.7
2	**270**	**1.00**	**63.0**
3	315	0.95	59.9
4	0	0.85	53.6
5	45	0.70	44.1
6	90	0.70	44.1
7	135	0.70	44.1
8	180	0.75	47.3

Storm Tide: The storm tide per Figure 3-5 [3] is 3.5 feet for all directions. This is the sum of storm surge and astronomical tide. Thus, the storm water depth for the benchmark platform is 160.5 feet (=157 feet + 3.5 feet).

Current: The current associated with the wave height for any given direction is a vector quantity and will depend on storm water depth (MLLW + storm tide) and platform longitude. The water depth of 160.5 feet places the current in the "intermediate zone" defined as 150 feet to 300 feet (Sec 2.3.4c4 of [3]). To obtain the surface current, linear interpolation is needed between the "Shallow Water Zone" and "Deep Water Zone" currents. The procedure for interpolation is given by example in Reference 3 ("Commentary on Hydrodynamic Force Guidelines, Section C2.3.4," page 123).

Shallow Water Zone Current: The longitude of the platform is 91.33 degrees. The surface current is a vector with a magnitude of 2.1 knots (3.55 ft/sec). Its direction, based on Figure 3-6 [3] is at 280 degrees. For interpolation, the water depth is taken as 150 feet.

Deep Water Zone Current: In deep water only, the components of the current in the direction of the wave are important and the transverse current is negligible. According to Sec 2.3.4c4 of Reference 3, the magnitude of the surface current in the principal wave direction (290 degrees) is 2.1 knots. The magnitudes of the current for the rest of the wave directions in Figure 3-1 are obtained by applying, to the 290-degree current, the same factors that are applied to the wave heights (Figure 3-4). This current is assumed to apply to the given direction of ±22.5 degrees. For interpolation, the water depth is taken as 300 feet.

Interpolated Current at Platform Location: The interpolated in-line and transverse currents for a water depth of 160.5 feet are given below. A negative in-line current means that the in-line component of the current opposes the wave. Transverse current is the component of the current that is normal to the in-line current. The current direction is measured counterclockwise with respect to the wave direction.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Factors to apply to Omni-directional Current	Interpolated In-line Current (knots)	Interpolated Transverse Current (knots)	Applicable In-line Current (knots)	In-line current at 150 ft. (knots)	In-line current at 300 ft. (knots)
1	225	0.90	1.25	-1.60	1.25	1.2	1.89
2	**270**	**1.00**	2.07	-0.34	**2.07**	**2.07**	**2.10**
3	315	0.95	1.74	1.12	1.74	1.72	2.00
4	0	0.85	0.46	1.92	0.46	0.36	1.79
5	45	0.70	-1.02	1.60	0.20	-1.20	1.47
6	90	0.70	-1.82	0.34	0.20	-2.06	1.47
7	135	0.70	-1.50	-1.12	0.20	-1.72	1.47
8	180	0.75	-0.23	-1.92	0.20	-0.36	1.58

In performing the interpolation, the API TG notes that the example in the RP 2A Commentary is not consistent with the intent of the main text. Specifically, the check on whether or not the in-

line current (greater than or equal to 0.2 knots) should be performed *after* interpolation, not *prior* to interpolation as implied by the Commentary. From a practical point of view, the sequence will not be too important for the most forceful waves. However, for consistency and validity of forces for all directions, the check should be performed *after* interpolation. The example will be corrected in the upcoming 21st edition.

Current for Design Level Guideline Forces: The appropriate surface current for calculating the 20th edition design level guideline forces is given in the column labeled "Applicable In-line Current" of the above table. This is the same as the in-line current in Column 4, except that a minimum speed of 0.2 knots is used (Sec 2.3.4c4 of [3]). The current profile is uniform over the water column per Figure 3-7 [3].

It is believed that it is sufficient to use the in-line current for analysis. However, it is acceptable to include the transverse component of the current, given in Column 5 of the above table, provided the specified vector current is consistent with the in-line component given in Column 6. This issue will receive further attention by the API Task Group on Wave Force Commentary, and a clarification will be provided in the 21st edition of API RP2A.

Wave Period: The wave period is 13 seconds for all directions (Sec 2.3.34c5 of [3]). This is the period measured at a fixed point. For the purpose of obtaining wave kinematics that may be superimposed on the in-line current, the apparent wave period (T_{app}, period measured in a coordinate system with the wave) is needed and is given below. It is based on the applicable in-line current given in the above table and is calculated using Figure 3-8 [3].

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Applicable In-line Current (knots)	Apparent Wave Period T_{app} (sec)
1	225	1.25	13.5
2	**270**	**2.07**	**13.8**
3	315	1.74	13.7
4	0	0.46	13.2
5	45	0.20	13.1
6	90	0.20	13.1
7	135	0.20	13.1
8	180	0.20	13.1

Wind Speed: The one-hour wind speed at an elevation of 10 meters (32.81 feet) is 80 knots (Sec 2.3.4c7 of [3]). Wind is colinear with wave directions.

Marine Growth: The marine growth thickness is 1.5 inches and extends from Elevation (+)1 foot to (-)150 feet (Sec 2.3.4d2 of [3]).

3.2 Applicable Section 17 Criteria

For the benchmark analysis, the platform was categorized as "Manned Evacuated" with "Significant Environmental Impact." Therefore, the "FULL POPULATION HURRICANE" metocean criteria per Fig. 1-1 was applicable. The basic criteria given in Table 17.6.2-1 of Section 17 [1], for full population hurricane, is as follows:

Metocean Criteria	Units	Design Level Analysis	Ultimate Strength Analysis
Wave Height and Storm Tide	ft	per Fig. 3-9	per Fig. 3-9
Deck Height	ft	per Fig. 3-10	per Fig. 3-10
Wave & Current Direction	-	Omnidirection	20th Ed.
Current Speed	knots	1.6	2.3
Wave Period	sec	12.1	13.5
Wind Speed (1-hr @ 10m)	knots	65	85

3.2.1 Design Level Metocean Criteria

Wave Heights: The omnidirectional wave height per Figure 3-9 [1] is 55 feet. Wave heights, as a function of the required (for force calculation) wave direction, are given in Column 5 of the following table. The wave heights were obtained by choosing, for each direction, the lower value of the 55-foot wave height and the 20th edition wave height, as assessment design level criteria should not be larger than the basic 20th edition criteria.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Omnidirectional Design Level Wave Height (ft)	20th Edition Directional Wave Height (ft)	Applicable Section 17 Design Level Wave Height (ft)
1	225	55.0	56.7	55.0
2	**270**	**55.0**	**63.0**	**55.0**
3	315	55.0	59.9	55.0
4	0	55.0	53.6	53.6
5	45	55.0	44.1	44.1
6	90	55.0	44.1	44.1
7	135	55.0	44.1	44.1
8	180	55.0	47.3	47.3

Storm Tide: The storm tide per Figure 3-9 [1] is 3.0 feet for all directions. This is the sum of storm surge and astronomical tide. The storm water depth for the benchmark platform is 160 feet (=157 feet + 3 feet).

Current: The appropriate surface current given in Table 17.6.2-1 of Section 17 [1] is summarized below. The currents were obtained by choosing, for each direction, the lower value of 1.6 knots (see summary table in Sec. 3.2) and the 20th edition current (per Section 3.1).

The current profile is uniform over the water column as shown in Figure 3-7.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	In-line Current (knots)	Transverse Current (knots)	20th Edition In-line Current (knots)	Applicable Section 17 Design Level In-line Current (knots)
1	225	1.6	0.0	1.25	1.25
2	**270**	1.6	0.0	2.07	**1.60**
3	315	1.6	0.0	1.74	1.60
4	0	1.6	0.0	0.46	0.46
5	45	1.6	0.0	0.20	0.20
6	90	1.6	0.0	0.20	0.20
7	135	1.6	0.0	0.20	0.20
8	180	1.6	0.0	0.20	0.20

Wave Period: The apparent wave period, T_{app} is shown below. It is based on the design level in-line current in Column 3 and is obtained using Figure 3-8 (Figure 2.3.1-2 of [3]). The basic wave period is 12.1 seconds.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Applicable In-line Current (knots)	Apparent Wave Period, T_{app} (sec)
1	225	1.25	12.5
2	**270**	1.60	**12.6**
3	315	1.60	12.6
4	0	0.46	12.3
5	45	0.20	12.2
6	90	0.20	12.2
7	135	0.20	12.2
8	180	0.20	12.2

Wind Speed: The one-hour wind speed at an elevation of 10 meters is 65 knots per summary table given in Section 3.2. Wind is colinear with wave directions.

Marine Growth: The marine growth thickness is 1.5 inches and extends from Elevation (+)1 foot to (-)150 feet per Sec 2.3.4d2 of Reference 3.

3.2.2 Ultimate Strength Metocean Criteria

Wave Heights: The omnidirectional wave height per Figure 3-9 is 68 feet [1]. Wave heights, as a function of the required (for force calculation) wave direction, are given below. The wave heights were obtained by applying the same factors that were applied to arrive at the 20th edition wave heights.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Factors to apply to the Omni-directional Wave Height	Directional Wave Height (ft)
1	225	0.90	61.2
2	**270**	**1.00**	**68.0**
3	315	0.95	64.6
4	0	0.85	57.8
5	45	0.70	47.6
6	90	0.70	47.6
7	135	0.70	47.6
8	180	0.75	51.0

Storm Tide: The storm tide per Figure 3-9 [1] is 3.0 feet for all directions. This is the sum of storm surge and astronomical tide. The storm water depth for the benchmark platform is 160 feet (=157 feet + 3 feet).

Current: The appropriate surface current is given below. The currents were obtained using the same procedure that was used for the 20th edition currents. The current magnitude is 2.3 knots at 280 degrees from True North as given in summary table in Section 3.2, as opposed to the 2.1 knots in the 20th edition.

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Factors to apply to the Omni-directional Current	In-line current at 150 ft. (knots)	In-line current at 300 ft. (knots)	Interpolated In-line Current (knots)	Interpolated Transverse Current (knots)	Applicable Ultimate Strength In-line Current (knots)
1	225	0.90	1.32	2.07	1.37	-1.76	1.37
2	**270**	**1.00**	**2.26**	**2.30**	2.27	-0.37	**2.27**
3	315	0.95	1.88	2.18	1.90	1.23	1.90
4	0	0.85	0.40	1.95	0.50	2.11	0.50
5	45	0.70	-1.32	1.61	-1.12	1.76	0.20
6	90	0.70	-2.26	1.61	-2.01	0.37	0.20
7	135	0.70	-1.88	1.61	-1.65	-1.23	0.20
8	180	0.75	-0.40	1.72	-0.26	-2.11	0.20

The current profile is uniform over the water column per Fig. 3-7 (Fig 2.3.4-6 [3]).

Wave Period: The apparent wave period, T_{app} is given in the following table. It is based on the in-line current in different directions given above, with basic wave period of 13.5 seconds, and is calculated using Figure 3-8 [1].

Direction Number	Wave Direction Toward, Clockwise from North (degrees)	Applicable In-line Current (knots)	Apparent Wave Period, T_{app} (sec)
1	225	1.37	14.0
2	**270**	2.27	**14.4**
3	315	1.90	14.2
4	0	0.50	13.7
5	45	0.20	13.6
6	90	0.20	13.6
7	135	0.20	13.6
8	180	0.20	13.6

Wind Speed: The one-hour wind speed at an elevation of 10 meters is 85 knots (per Table 17.6.2-1 of [1]). Wind is colinear with wave directions.

Marine Growth: The marine growth thickness is 1.5 inches and extends from Elevations (+)1 foot to (-)150 feet (per Sec 2.3.4d2 of [3]).

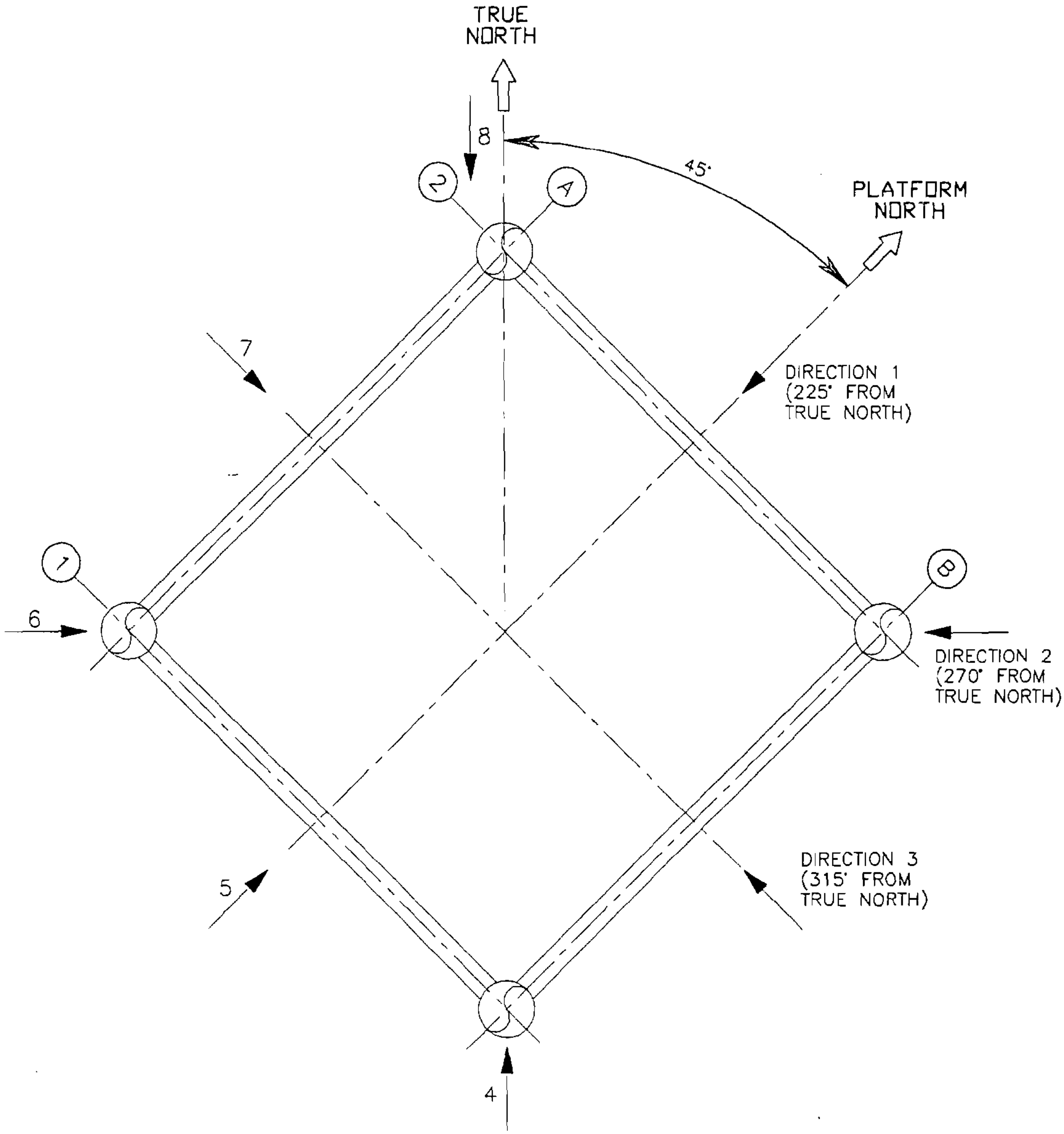


Figure 3–1: Wave Approach Directions

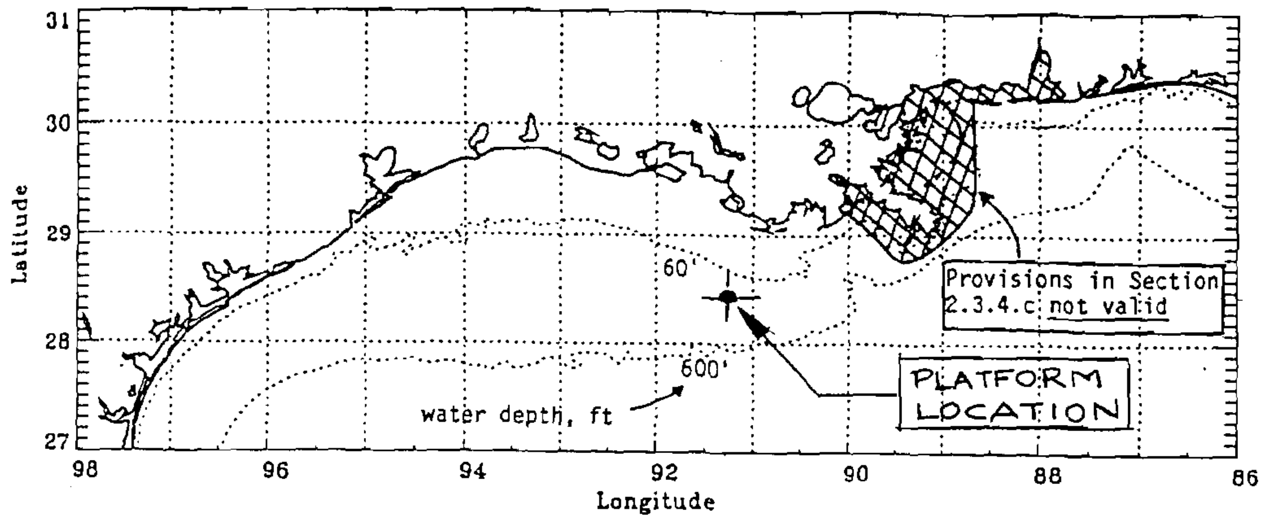


Figure 3-2 Region of Applicability of Extreme Metocean Criteria in API RP 2A, 20th Edition (Refer Fig. 2.3.4-2 of API RP 2A, 20th Ed.)

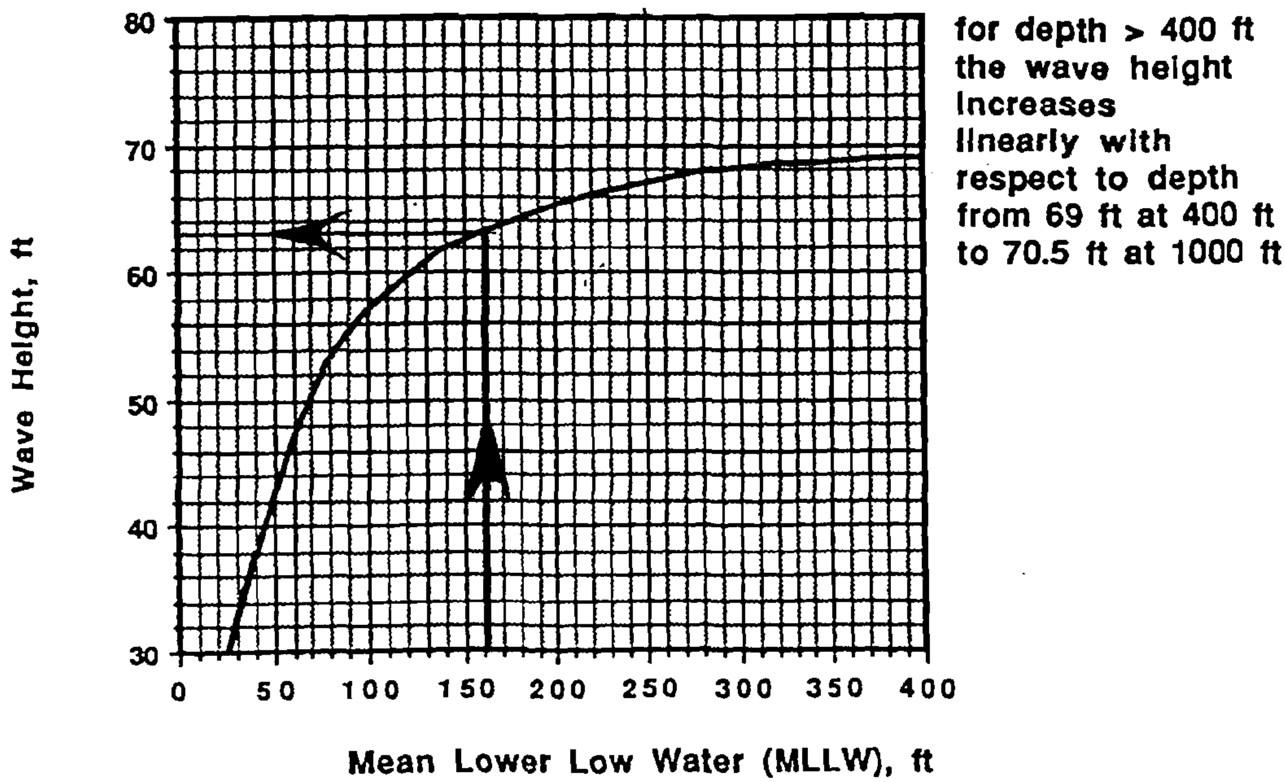


Figure 3-3 Guideline Omni-directional Design Wave Height vs. MLLW, Gulf of Mexico, North of 27° N and West of 86° W (Refer Fig. 2.3.4-3 of API RP 2A, 20th Ed.)

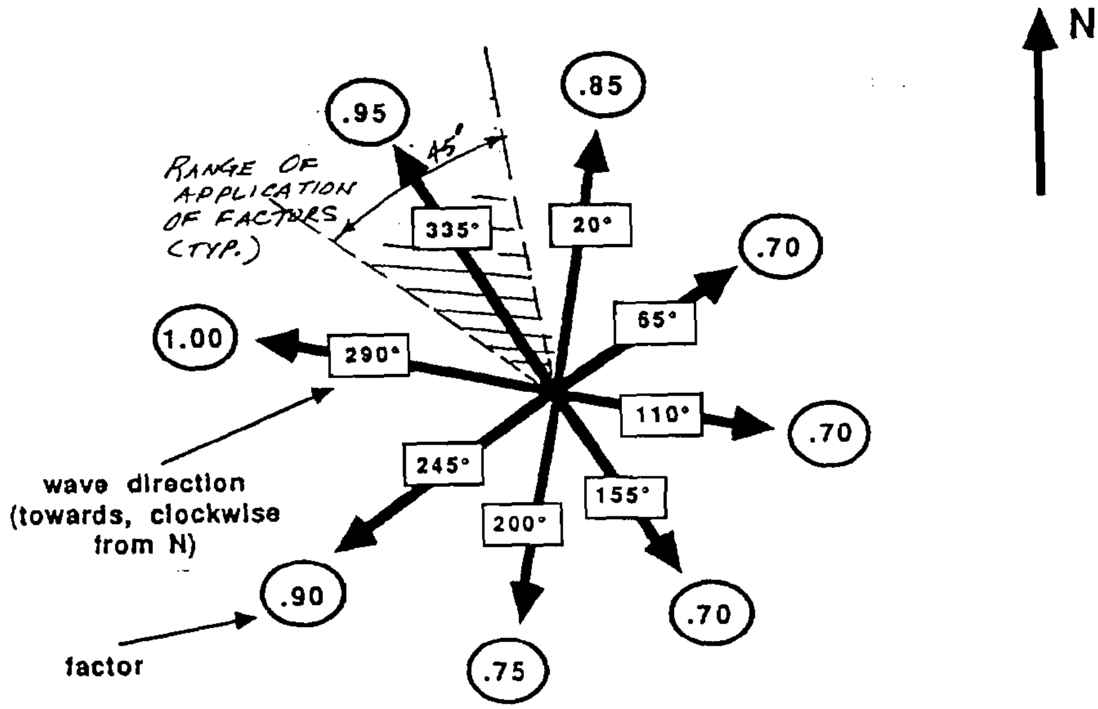


Figure 3-4 Guideline Design Wave Directions and Factors to Apply to the Omni-Directional Wave Heights (Fig. 3-3), Gulf of Mexico, North of 27° N and West of 86° W (Refer Fig. 2.3.4-4 of API RP 2A, 20th Ed.)

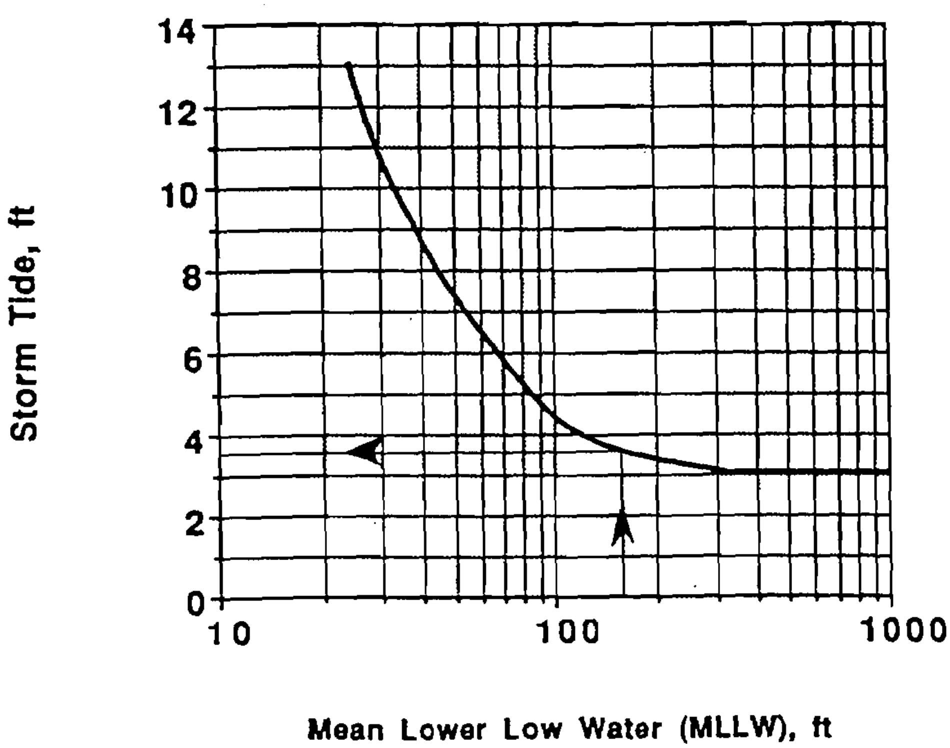


Figure 3-5 Guideline Design Storm Tide vs. MLLW, Gulf of Mexico, North of 27° N and West of 86° W (Refer Fig. 2.3.4-7 of API RP 2A, 20th Ed.)

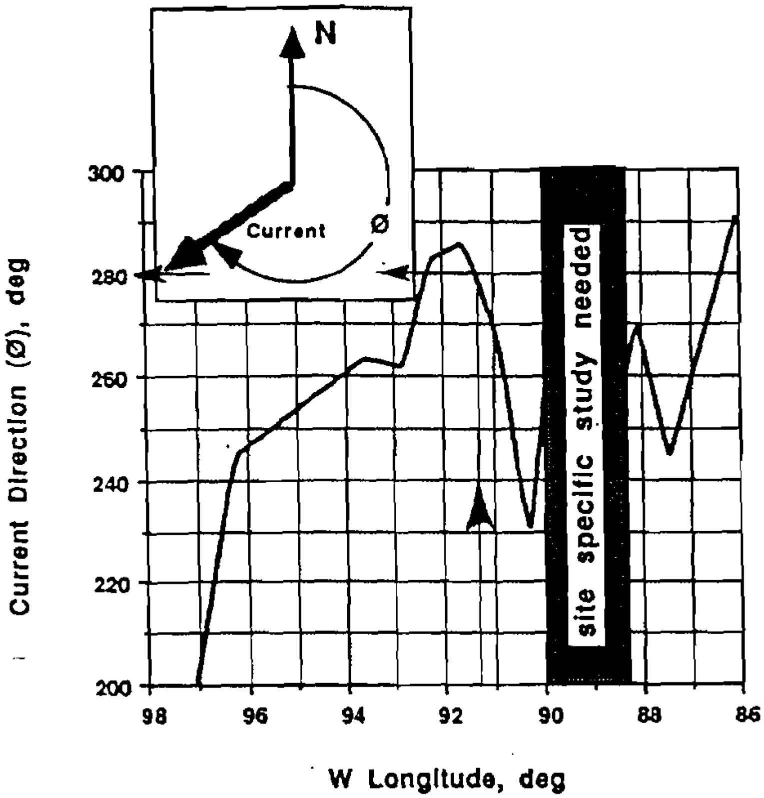


Figure 3-6 Guideline Design Current Direction (towards) with respect to North in Shallow Water (Depth < 150 ft), Gulf of Mexico, North of 27° N and West of 86° W (Refer Fig. 2.3.4-5 of API RP 2A, 20th Ed.)

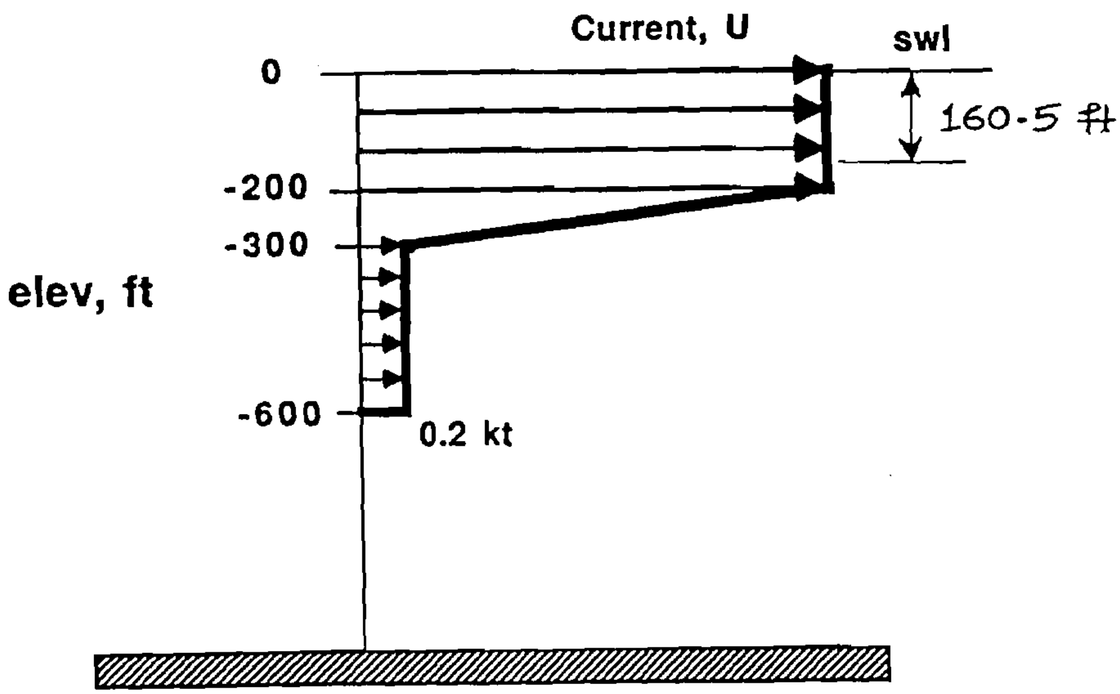


Figure 3-7 Guideline Design Current Profile, Gulf of Mexico, North of 27° N and West of 86° W (Refer Fig. 2.3.4-6 of API RP 2A, 20th Ed.)

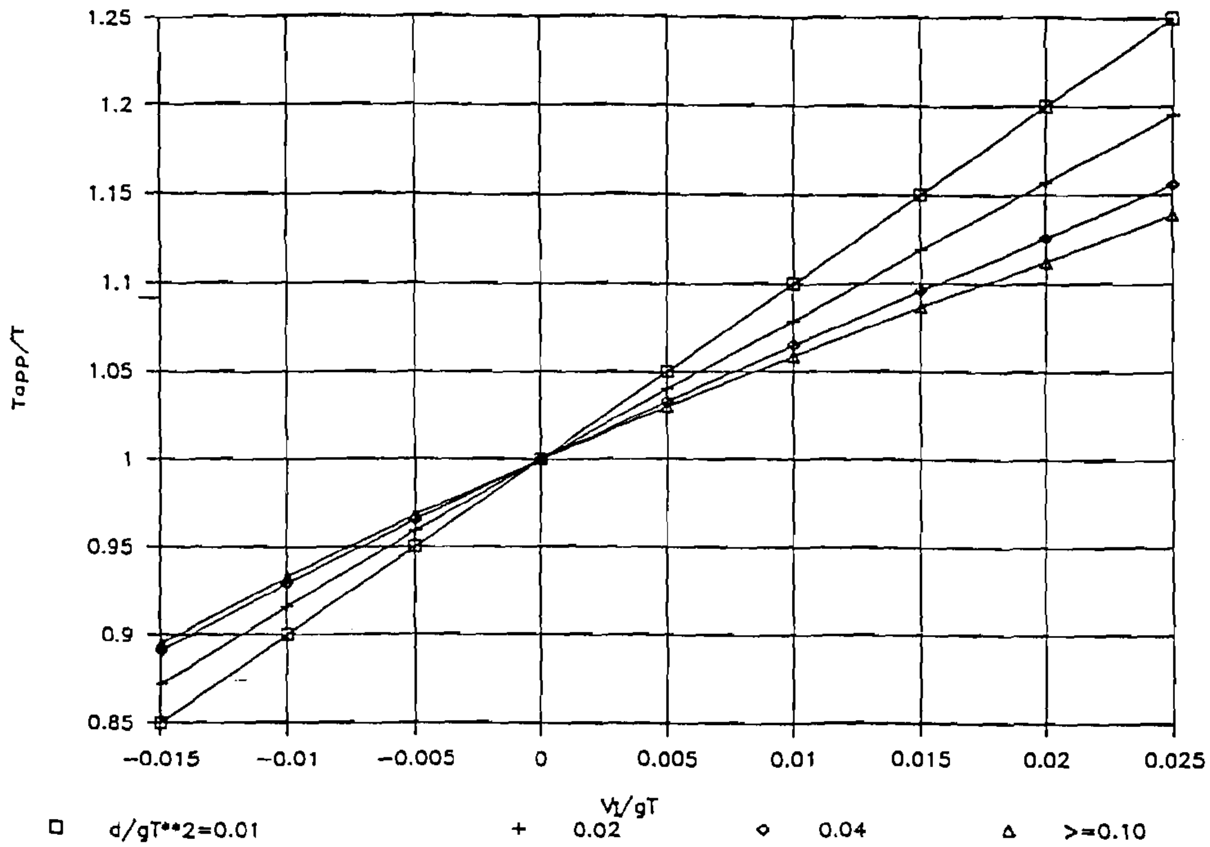


Figure 3-8 Doppler Shift due to Steady Current (Refer Fig. 2.3.1-2 of API RP 2A, 20th Ed.)

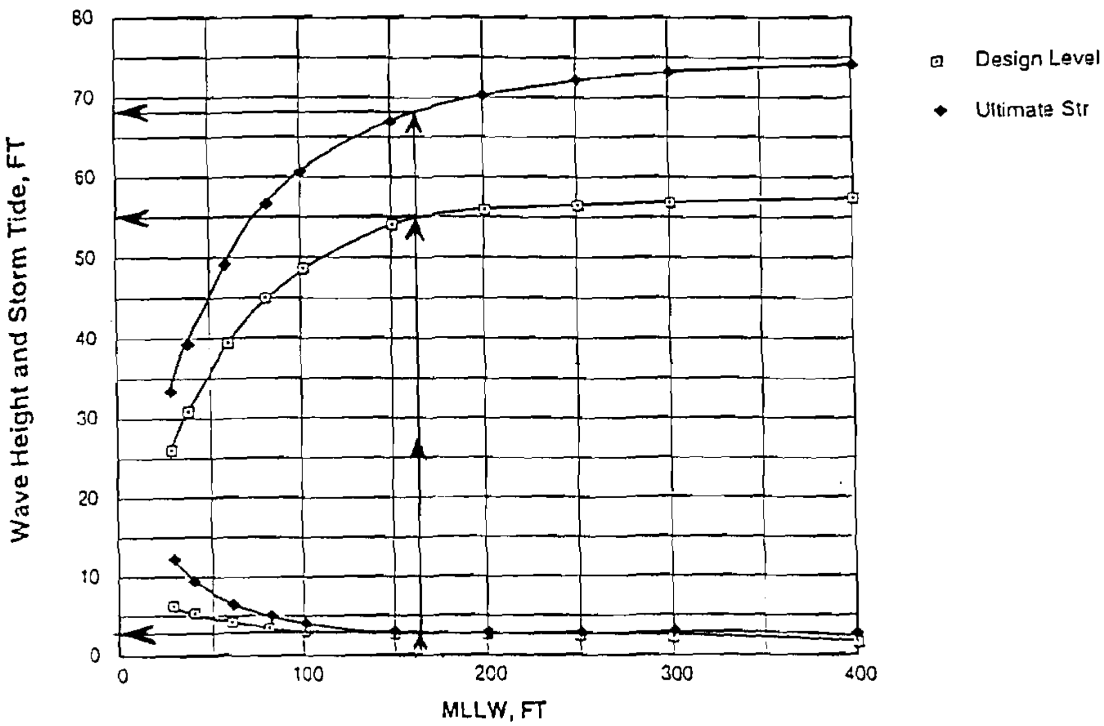


Figure 3-9 Full Population Hurricane Wave Height and Storm Tide Criteria (Refer Fig. 17.6.2-2a of API RP 2A, 20th Ed.)

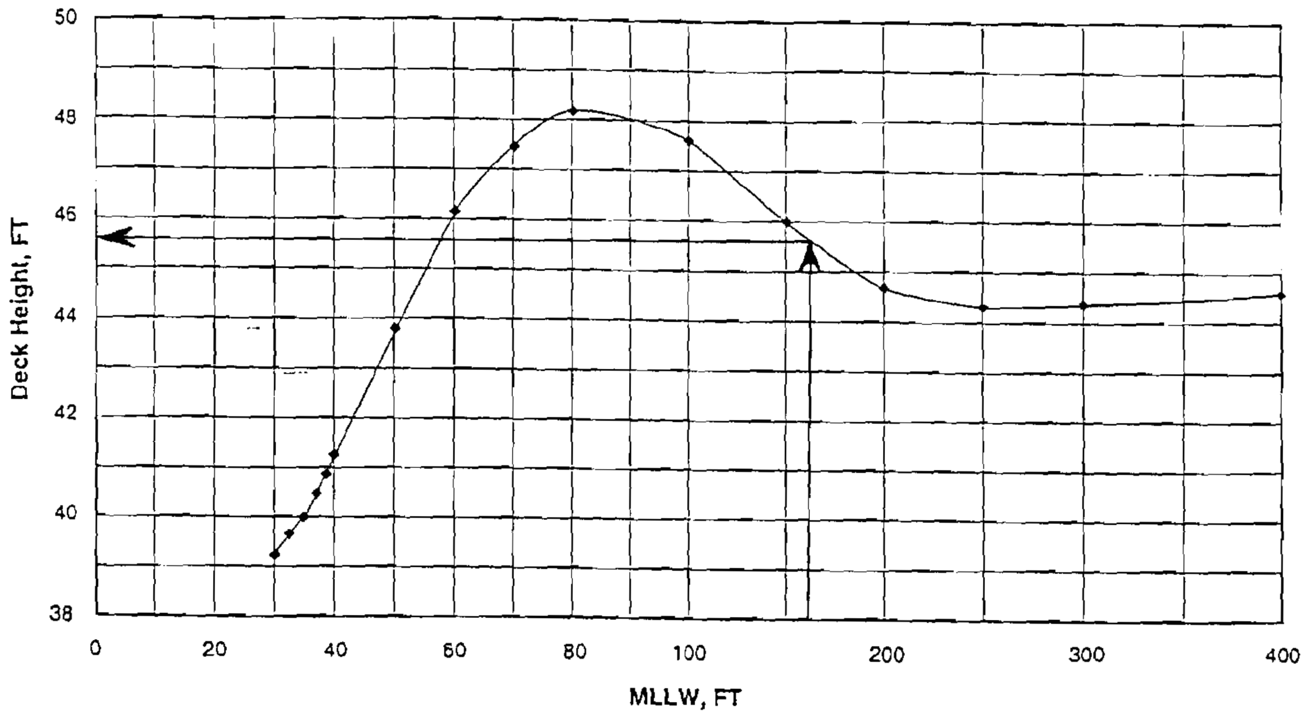


Figure 3-10 Full Population Deck Height Criteria (Refer Fig. 17.6.2-2b of Supplement 1 API RP 2A, 20th Ed.)

4.1 Introduction

A 3-dimensional computer model was developed for the static pushover analysis to determine the ultimate strength of the platform. In the ultimate strength analysis, the nonlinear behavior of elements (i.e., legs, braces, joints, piles) up to and beyond their ultimate strengths are modeled. In comparison, all structural elements in a linear elastic design level analysis, are modeled as linear beam elements, and generally the soil-structure interaction is explicitly modeled by nonlinear springs (some companies model equivalent linear pile/soil model).

To determine the ultimate strength of a platform, the nonlinear behavior of all elements, which are likely to yield or fail against overloading, are modeled. Additionally, the best estimates of the strengths and stiffnesses of various elements (i.e., braces, joints) and material properties (yield strength, soil strength) are used in an ultimate strength analysis to predict their most likely behavior under extreme loading, instead of the lower bound estimates used generally in a design level analysis.

The various elements are modeled to include the following properties and behaviors: material yield and ultimate strength, post-yield behavior, buckling, damage, and failure. Detailed discussion on various types of nonlinear elements used in the pushover method of analysis is given in Commentary C17.7.3c in Section 17 and other literature [1, 7]. This section presents the modeling assumptions that were used to develop the model shown in Figure 4-1 for pushover analysis.

4.2 Basic Assumptions

A basic consideration in the development of the ultimate strength analysis model was to remove the factors of safety and other known sources of conservative bias in characterizing the strengths of the various elements. In general, the mean estimates of the material properties and element strengths were modeled.

The benchmark platform was assumed to have no damage or deterioration of its primary and secondary structural elements.

All material was assumed to be of A36 steel, with a nominal yield strength of 36 ksi. A mean yield strength of 42 ksi was used in the analysis to account primarily for the increase from nominal to mean strength. Mill certificates and coupon tests were not available for this platform.

The structural elements that are likely to yield or fail due to overloading were modeled by nonlinear elements to represent their material and geometrical nonlinearities. Explicit modeling of joint cans may be needed for platforms with failure modes (yielding, hinging) in jacket legs.

The following element behaviors were considered in the development of the model shown in Figure 4-1:

Element Description	Element Behavior
Deck elements (1st level to 4th level)	Linear elastic
Deck legs below 1st level deck	Nonlinear
Jacket elements - legs, braces, joints	Nonlinear
Conductors, risers, pump casings, sump caissons	Wave load elements
Boat landings and barge bumpers	Equivalent wave load elements
Piles	Nonlinear
Soil/Pile interaction	Nonlinear

Several other appurtenances and attachments that neither impose significant loading nor provide strength to the primary jacket and piles were not modeled. Such items were ladders from the jacket to the deck, grāting and handrails, conductor guides and plating, and anodes. However, for a particular platform, especially with damage and/or deterioration, it may be important to include the effects of some of these appurtenances.

4.3 Nonlinear Element Types

The element type used to represent a structural member depends on its expected behavior under extreme loading conditions. The nonlinear elements discussed in references cited [1,7] include lumped-plasticity and distributed-plasticity beam-columns, struts, nonlinear truss bars, damaged members, near-field soil elements, gap-friction elements, and shim elements. The API documents and other related literature should be consulted for theoretical details and the formulations used to define these elements.

In this section, various nonlinear elements for the deck, jacket, pile, and soil used in the development of the model shown in Figure 4-1 are identified. A summary of the structural elements and the nonlinear elements used to represent the various failure modes is given in the table below, followed by further discussion.

Structural Elements	Failure Modes Modeled	Element Type
Deck legs	Yielding/hinging of leg section	Beam-Column
Jacket legs	Yielding/hinging of leg section	Beam-Column
Jacket primary braces	Buckling/tensile yielding of braces	Strut
	Yielding/hinging of brace section	Beam-Column
Jacket leg/pile annulus at horizontal frame levels	Lateral load transfer	Shim elements
Piles	Yielding/hinging of pile section	Beam-Column
Soil/Pile interaction	Overloading beyond soil capacity (lateral, axial, end bearing)	p-y, t-z, q-z curves

All other structural elements not identified above were modeled either as linear elastic beam or as wave load elements (elements used in wave load analysis and excluded in the stiffness analysis).

Deck Structure:

The deck legs from the bottom of the lower deck (deck level 1) to the top of the jacket were modeled as beam columns to capture yielding/hinging of leg sections and formation of the portal failure mechanism. The remaining deck framing (legs between deck levels 1 and 4, main girders, and braces) were modeled as linear elastic beams. The secondary girders, floor beams, and plating were modeled by horizontal X-braces to provide rigid body lateral load transfer and to minimize the size of the computer model.

For a particular platform, additional deck legs and girders may be modeled as nonlinear elements, if such a behavior is expected.

Jacket Structure:

Legs: The jacket legs were modeled as beam-columns. The leg/brace joint cans were explicitly modeled by introducing additional nodes/elements between horizontal frame levels. Shim elements (with lateral load transfer capability) were introduced at all leg/horizontal framing nodes from Elevations (-)8′ to (-)157′, and also at the bottom of the leg extension at Elevation (-)169′. Additional overturning resulting from large lateral displacements and vertical loads (i.e., the P-delta effect) was included.

Braces/Joints: When capacities of braces and K-joints were checked, it was noted that joint capacity governed for the upper two bays (mean joint capacity was lower by 15%) and brace capacity governed at all other locations. However, the joint behavior was not modeled, due to larger biases and uncertainties in their formulations. In an actual application, the joint behavior would be included [10].

The buckling capacity of a brace was defined by Equation D.2.2-2 of API RP 2A, LRFD [9]. The brace capacity was modified to account for the effect of lateral wave and current forces.

The effective length (k) factors for "K," "diagonal," and "X" bracing were taken as 0.65 instead of the larger "k" values given in the API RP 2A [3]. These lower values were based on results of recent laboratory tests [11,12] and analytical studies [13]. The length was taken as node-to-node in the computer model (not face-to-face of the legs). For X-braces, the member lengths were taken as one-half the node-to-node lengths (i.e., out-of-plane buckling is restricted due to the compensating effect of the tension brace).

Conductors: The conductors were modeled as wave load elements and were not modeled to include their contribution to strength and stiffness. In some cases, conductors may be modeled as structural elements to include their contribution to the lateral capacity of the foundation and the jacket.

Pile Foundation:

The piles were modeled by a number of beam-column elements with varying lengths and sizes to represent the pile makeup, and to also account for the load transfer mechanisms involved from the jacket/deck to the piles and from the piles to soil. The piles above the mudline were modeled by introducing additional nodes at wall thickness changes, at horizontal framing levels where shim elements are modeled, and at the jacket-to-pile connections at Elevation (+)13′. The pile element lengths below the mudline were defined to represent the pile lateral load deformation behavior to a reasonable level of activity. The variation in soil strata was also used in defining the pile nodes.

Soil/Structure Interaction:

The soil resistance was represented by soil springs, which are characterized by nonlinear p-y (lateral bearing), t-z (axial shaft skin friction), and q-z (end bearing) curves given in API RP 2A, 20th Edition. These springs are also called near-field elements that connect the piles to the soil, vertically and horizontally [7]. These elements were distributed along the length of each pile and represented the forces generated at the interface between the soil and pile for a variable tributary length of pile. The hysteretic behavior was not modeled.

The soil profile is shown in Figure 2-18, and the locations of soil springs are shown in Figure 4-1. There should be at least one soil spring representing each soil layer. In these soil springs, only translational load deflection curves are typically included in the pile analysis, while rotational load deflection relationships are neglected.

Lateral p-y springs: The lateral p-y nonlinear springs, attached to the pile nodes, were modeled using the static capacity estimates given in API RP 2A as opposed to the cyclic p-y springs used in a new design. Recent centrifuge model tests [14] have indicated that for pushover analysis, the static lateral soil capacity provides a better ultimate strength prediction. This is because the displacements of piles at ultimate loading are significantly greater than the typical test displacements on which the API p-y behavior is based.

Axial t-z springs: Nonlinear t-z springs attached to the pile nodes were based on static axial capacity per API RP 2A, with no reduction in the capacity. The effect of pile flexibility (pile length effect) was explicitly accounted for in the analysis, by using the PSAS suite of elements (nonlinear soil spring elements used with the CAP software), which model the loss of skin friction at large pile displacements. The contributions of other factors, such as loading rate (or strain rate effect), cyclic loading, reconsolidation (time effect), and aging effect, which would vary the basic API static capacity estimate were not considered in this analysis [15,16]. No corrections for soil strength bias [16] were included in this analysis.

The piles were modeled as plugged, thus the pile interior-soil friction was not considered.

4.4 Wave Load Elements

Several appurtenances and secondary structures were modeled as wave load elements to evaluate metocean loads only and were excluded in the capacity analysis. Details of these elements given are not intended to represent the most correct way to model these structures (such as boat landing), but are given to enable one to identify the sources for differences in their analysis results from those given in this document. The walkways and handrails were not modeled.

Boat Landings: Each of the boat landings was modeled by a 30 ft by 13 ft rectangle with equivalent horizontal and vertical members to define the wave/current loads. The details of equivalent elements used are:

Vertical members		
Number of members	=	2
Diameter of members	=	2 ft
Length of members	=	13 ft
Equivalent C_d	=	2.11
Horizontal members:		
Number of members	=	2
Diameter of members	=	2 ft
Length of members	=	30 ft
Equivalent C_d	=	0.66

The boat landing models are attached to the Row 1 and Row B frames (Figures 2-13 and 2-15).

In some cases, the boat landings may be modeled more explicitly or more refined modeling techniques may be used.

Barge Bumpers: Each barge bumper (Figure 2-16) was modeled by an equivalent member with the following properties:

Equivalent diameter	=	3.33 ft
Length of member	=	18 ft
Equivalent C_d	=	0.72

The barge bumpers are attached to the Row 1 and Row B frames (Figures 2-13 and 2-15).

Conductors: Four conductors were modeled individually at their exact locations spanning from the lower deck to the mudline. Three conductors were modeled as 30-inch diameter tubulars and one conductor as a 48-inch diameter tubular. The same marine growth on the jacket structure was considered for the conductors.

<u>Risers:</u> Eight risers span from Elevation (+) 43′ to mudline. All risers were considered to be supported laterally at all the horizontal framing levels from Elevations (+)10′ to (-)126′. A 0.5-inch corrosion coating was assumed for all risers.

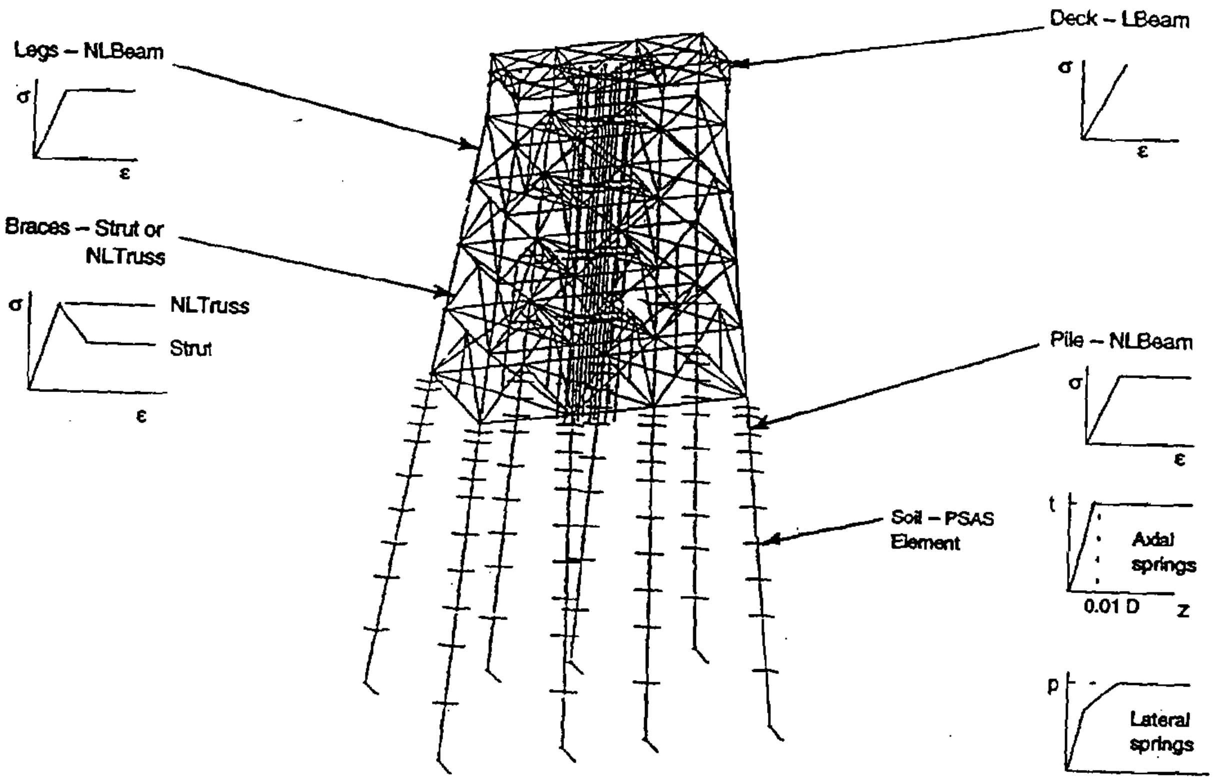


Figure 4-1 Nonlinear Computer Model for Ultimate Capacity Analysis

5.1 Load Analysis Procedure

The API recommended procedure for computation of deterministic static wave forces on a fixed platform (neglecting platform dynamic response and distortion of incident wave by the platform) given in Figure 2.3.1-1 of API RP 2A was used (Figure 5.1). The stepwise wave and current force computation procedure is discussed in detail in Section 2.3.1 of API RP 2A, and the wind load computation procedure is discussed in Section 2.3.2. Further discussion of the basis used to develop the RP2A guidelines can be found in Reference 17.

The total lateral and overturning design and ultimate metocean loads consist of the following:

- Wind load on the deck and jacket
- Wave/current loads on the jacket structure and all other components
- Wave/current loads on the deck

In this section, the values selected for various parameters for the benchmark platform are given and the procedures used are discussed. The metocean loads are provided for the three wave approach directions (225°, 270°, and 315° from True North) that would result in higher loading.

The metocean load analysis was performed for three storm approach directions described above. The metocean criteria identified in Section 3 for these directions for three analysis cases (API 20th Edition, Section 17 design level, and Section 17 ultimate strength level) are summarized as follows:

Metocean Criteria	Direc No.	Approach Direction from True North	Storm Tide (ft)	Wind Speed (knots)	Wave Height (ft)	Apparent Wave Period (sec)	In-Line Current Speed (#1) (ft/sec)
RP2A, 20th Edition	1	225°	3.5	80	56.7	13.5	2.11
	2	270°	3.5	80	63.0	13.8	3.50
	3	315°	3.5	80	59.9	13.7	2.94
Section 17, Design Level	1	225°	3.0	65	55.0	12.5	2.11
	2	270°	3.0	65	55.0	12.6	2.70
	3	315°	3.0	65	55.0	12.6	2.70
Section 17, Ultimate Strength	1	225°	3.0	85	61.2	14.0	2.32
	2	270°	3.0	85	68.0	14.4	3.83
	3	315°	3.0	85	64.6	14.2	3.21

Notes: (#1) - Current in-line with the wave

5.2 Wave/Current Load Calculation Procedure

The following parameters/factors were determined from Section 2 of API RP 2A, 20th Edition, and are applicable to all three analysis cases:

Wave Kinematics Factor: The wave kinematics factor is taken as 0.88 for hurricane condition (Sec 2.3.4d1 and 17.6.2a of RP 2A).

Current Blockage Factor: The platform has four legs and is considered to be a "typical" jacket-type structure. The current blockage factor is 0.80 for end-on and broadside directions and 0.85 for diagonal directions (Sec 2.3.1b4 of RP 2A). The blockage factor should be applied to the in-line current given in Column 8 of the table provided in Section 5.1.

Force Coefficients: Design waves for the Gulf of Mexico that are associated with the most forceful directions are usually sufficiently high so that default values of the force coefficients will apply. For other directions, the waves may be small enough that the force coefficients need to consider wake encounter effects. However, those directions may not control the design and are usually ignored.

A simple measure of whether or not default values are applicable by determining the value of $U_{mo}\ T_{app}/D$, where U_{mo} is the maximum horizontal wave particle velocity at the storm water level, T_{app} is the apparent wave period, and D is the diameter of platform leg at the storm water level (Sec 2.3.1b7 of RP 2A).

If $U_{mo}\ T_{app}/D \geq 30$ Use default values for the force coefficients as follows:
Smooth tubular: $C_d = 0.65$; $C_m = 1.6$
Rough tubular: $C_d = 1.05$; $C_m = 1.2$

If $U_{mo}\ T_{app}/D < 30$ Determine appropriate coefficients as per the procedure given in the Commentary to Section 2 of RP 2A.

For the benchmark analysis, the default force coefficients values were assumed to be applicable to all load cases.

Wave Theory: The applicability of a wave theory to a platform and metocean parameters is determined from Figure 5-2 [3]. In this case, the 9th Order Stream Function wave theory is applicable for all analysis cases.

Other wave theories such as Extended Velocity Potential and Chappelear may be used if an appropriate order of solution is selected.

Conductor Shielding Factor: The shielding factor for wave loads on conductor arrays as a function of conductor spacing is determined from Figure 5-3 [3]. The conductor shielding was

ignored (i.e., used shielding factor = 1.0) because there are only four conductors and the spacing is irregular.

5.3 Wind Load

Equation 2.3.2-8 in RP 2A was used to compute the wind forces on the jacket, deck structure, and equipment. In the case of pushover analysis, the global wind force was computed using the mean wind speed averaged over an hour. The gust factor and local wind effects, such as pressure concentrations and internal pressures were not used.

The wind speed obtained for the 100-year return period case and for Section 17 design level and ultimate strength cases are given in the table in Section 5.1.

The projected areas for the deck used are given in Section 2.1.3 and Figure 2-11. The shape coefficient (C_s) was selected as 1.0 for "overall projected area of platform" (Section 2.3.2e of RP 2A) and was applied for wind load computations for all directions.

The computed wind loads for three analyses are given as follows:

Metocean Criteria	Direc. No.	Approach Direction from True North	Wind Speed at Elev. (+) 10m (ft/sec)	Projected Deck Area (#1) (ft^2)	Shape Coefficient C_s	Wind Force, F_{wind} (kips)	Wind OTM, M_{wind} (#2) (k-ft)
RP2A, 20th Edition	1	225°	135.20	2,081	1.0	52	11,470
	2	270°		3,070	1.0	78	17.273
	3	315°		2,261	1.0	57	12,594
Section 17, Design Level	1	225°	109.85	2,142	1.0	35	7,733
	2	270°		3,214	1.0	53	11,600
	3	315°		2,402	1.0	40	8,672
Section 17, Ultimate Strength	1	225°	143.65	2,923	1.0	84	18,850
	2	270°		1,981	1.0	57	12,488
	3	315°		2,153	1.0	62	13,724

Notes: (#1) - The projected areas vary due to variation in the wave crest elevations for different metocean criteria

(#2) - Overturning moment computed at Elevation (-)157′

5.4 Wave/Current Load on Jacket Structure

Total base shear and overturning moment were computed by vector summation of local drag and inertia forces on the jacket elements and deck legs due to wave and current. The local wave/current forces were computed using the Morison formula given in Equation 2.3.1-1 of API RP 2A. The slam, lift, and axial Froude-Krylov forces were neglected in this analysis.

The modeling of boat landings, barge bumpers, conductors, risers, etc. were discussed in Section 4. The wave/current loads on these attachments were also computed.

The wave crest was considered at several positions relative to the structure to determine the maximum base shear values. The base shear and overturning moment are provided for the wave crest position causing maximum base shear. The pushover load pattern used in the capacity analysis was based on the maximum base shear.

The computed wave/current loads for three analyses cases are given as follows:

Metocean Criteria	Direc No.	Approach Direction from True North	Wave Height (ft)	Apparent Wave Period (sec)	Current Speed (ft/sec)	Wave/Current Base Shear, F_{w-c} (kips)	Wave/Current OTM, M_{w-c} (k-ft)
RP2A, 20th Edition	1	225°	56.7	13.5	2.11	1,490	164,734
	2	270°	63.0	13.8	3.50	2,119	231,859
	3	315°	59.9	13.7	2.94	1,797	197,571
Section 17, Design Level	1	225°	55.0	12.5	2.11	1,325	152,324
	2	270°	55.0	12.6	2.70	1.423	160,955
	3	315°	55.0	12.6	2.70	1,413	160,450
Section 17, Ultimate Strength	1	225°	61.2	14.0	2.32	1,769	198,040
	2	270°	68.0	14.4	3.84	2,512	275,813
	3	315°	64.6	14.2	3.21	2,129	236,600

5.5 Wave/Current Load on Deck

The procedure for computation of the wave/current loads on the deck is given in Commentary C17.6 of Section 17 [1]. This procedure was calibrated to the deck forces measured in wave tank tests in which hurricane and winter storms were modeled.

The procedure is summarized in Figure 5-4. The steps used for computing the wave/current loads on deck were as follows:

- Determined wave crest elevation (h_c).
- Computed the wetted "silhouette" deck area (A) projected in the wave direction (θ_w).
- Calculated the maximum wave-induced horizontal fluid velocity (u_{wh}) at the wave crest elevation or the top of the main deck silhouette, whichever is lower.
- Identified the drag coefficient (C_{dk}) per Table C.17.6.2-1 [1] for the deck type.
- Computed wave/current force on the deck (F_{dk}), using wave kinematics factor (α_{wkf}) of 0.88 and current blockage factor (α_{cbf}) as given in Section 5-2.
- Applied the force (F_{dk}) at an elevation (Z_{dk}) above the bottom of the cellar deck, equal to 50% of the distance between the lowest point of the silhouette area and the wave crest or the top of the main deck, whichever is lower.

The wave crest height was computed using the 9th order Stream Function wave theory, determined in Section 5-2 based on the criteria given in Section 2.3.1b.2 of RP 2A, and the applicable wave height, associated wave period, and storm tide. The deck areas and plans are given in Figures 2-10 and 2-11 in Sections 2.1.2 and 2.1.3.

By this procedure, the magnitudes and the points-of-application of the horizontal deck forces were obtained. The values for the above parameters and the wave/current deck forces for three wave approach directions are given below. This deck force was added to the associated wave force on the jacket.

Metocean Criteria	Direc No.	Approach Direction from True North, θ_w	Wave Height (ft)	Wave Crest Elevation h_c (ft)	Wetted Silhouette Deck Area, A (ft^2)	Horiz. Fluid Velocity, u_{wh} (ft/sec)	Drag Coeff., C_{dk}	Current Blockage Factor, α_{cbf}
RP2A, 20th Edition	1	225°	56.7	38.07	60.87	21.33	2.0	0.80
	2	270°	63.0	42.88	166.46	24.44	1.5	0.85
	3	315°	59.9	40.50	88.26	22.84	2.0	0.80
Section 17, Design Level (See Note #2)	1	*225°*	*55.0*	*36.22*	*40.65*	*21.36*	*2.0*	*0.80*
	2	*270°*	*55.0*	*36.22*	*61.49*	*21.26*	*1.5*	*0.85*
	3	*315°*	*55.0*	*36.22*	*41.24*	*21.26*	*2.0*	*0.80*
Section 17, Ultimate Strength	1	225°	61.2	41.09	93.70	23.37	2.0	0.80
	2	270°	68.0	46.54	337.13	26.88	1.5	0.85
	3	315°	64.6	43.78	135.71	25.09	2.0	0.80

The computed wave/current loads for three analyses cases are given in the following table:

Metocean Criteria	Direc. No.	Approach Direction from True North	Wave Height (ft)	Current Speed (ft/sec)	Wave-in-Deck Base Shear, F_{dk} (kips)	Centroid of Force Above MSL, Z_{dk} (ft)	OTM Due to Wave-in-Deck, M_{dk} (#1) (k-ft)
RP2A, 20th Edition	1	225°	56.7	2.11	49	35.3	9,421
	2	270°	63.0	3.50	145	37.7	28,228
	3	315°	59.9	2.94	86	36.5	16,639
Section 17, Design Level (See Note #2)	1	*225°*	*55.0*	*2.11*	*33*	*34.3*	*6,314*
	2	*270°*	*55.0*	*2.70*	*39*	*34.3*	*7,462*
	3	*315°*	*55.0*	*2.70*	*35*	*34.3*	*6,697*
Section 17, Ultimate Strength	1	225°	61.2	2.32	91	36.8	17,634
	2	270°	68.0	3.84	355	39.5	69,759
	3	315°	64.6	3.21	159	38.1	31,025

Notes: (#1) - Overturning moment computed at Elevation (-)157′.
(#2) - The design level values are included for reference only. The design level check is inappropriate here as deck wave inundation occurs.

The above loads are based on the simple method recommended in Section 17, by which the variability of the deck force for a given wave height is rather large. The coefficient of variation (standard deviation divided by the mean) is about 0.35 [1]. Section 17 mentions that alternative procedures for static and/or dynamic analyses may be used provided they are validated with reliable and appropriate measurements of global wave/current forces on decks either in the laboratory or in the field.

5.6 Summary of Load Analysis Results

The total lateral loads used to develop the pushover load pattern includes the above three loads discussed in Sections 5.2 to 5.5, and are summarized as follows:

Lateral Loads -- Base Shear

Metocean Criteria	Approach Direction from True North	Wind Load on Deck (kips)	Wave/ Current Load on Jacket (kips)	Wave/ Current Load on Deck (kips)	Total Base Shear, BS (kips)
RP2A, 20th Edition	225°	52	1,490	49	1,591
	270°	78	2,119	145	2,342
	315°	57	1,797	86	1,940
Section 17, Design Level (See Note #2)	225°	*35*	*1,325*	*33*	*1,393*
	270°	*53*	*1,423*	*39*	*1,515*
	315°	*40*	*1,413*	*35*	*1,488*
Section 17, Ultimate Strength	225°	57	1,769	91	1,917
	270°	84	2,512	355	2,951
	315°	62	2,129	159	2,350

Lateral Loads -- Overturning Moment (#1)

Metocean Criteria	Approach Direction from True North	Moment from Wind Load on Deck (k-ft)	Moment from Wave/Current Loads on Jacket (k-ft)	Moment from Wave/Current Loads on Deck (k-ft)	Total Overturning Moment, OTM (k-ft)
RP2A, 20th Edition	225°	11,470	164,734	9,421	185,625
	270°	17,273	231,859	28,228	277,360
	315°	12,594	197,571	16,639	226,804
Section 17, Design Level (See Note #2)	225°	*7,733*	*152,324*	*6,314*	*166,371*
	270°	*11,600*	*160,955*	*7,462*	*180,017*
	315°	*8,672*	*160,450*	*6,697*	*175,819*
Section 17, Ultimate Strength	225°	12,488	198,040	17,634	228,162
	270°	18,850	275,813	69,759	364,422
	315°	13,724	236,600	31,025	281,349

Note: (#1) - Overturning moment computed at El. (-)157′
(#2) - The design level values are included for reference only. The design level check is not applicable here as deck wave innundation occurs.

5.7 Variations in JIP Participants' Results

The wave load analysis results initially submitted by thirteen participants in the JIP varied significantly when they independently identified the metocean criteria and hydrodynamic

parameters. The COV in the hydrodynamic loading (base shear) was about 24%. This included a few cases with incorrect metocean criteria used, which skewed the results. The participants resubmitted their results by removal of such "gross errors" and the COV in the base shear reduced to 12% [4].

API TG 92-5 performed a follow-up investigation [5] regarding loading with the voluntary cooperation of eleven of the organizations involved in the JIP. These participants were given the "correct" metocean criteria and parameters applicable to the benchmark platform, which were to be used in load computations. These quantities are identified in Section 2 of this document. A summary of the results obtained from this investigation is provided in Table 5.1.

The variations in the overall base shear forces for three metocean criteria [5] are as follows:

Load Type	Metocean Criteria	Mean Load (270 degree direction)	COV (270 degree direction)	Average COV (based on all 3 directions)
Wind Load	API 20th Edition	67	0.275	0.307
	API Section 17 - Design Level	46	0.285	
	API Section 17 - Ultimate Strength	76	0.292	
Wave/Current Loads on Jacket	API 20th Edition	2,277	0.089	0.087
	API Section 17 - Design Level	1,574	0.083	
	API Section 17 - Ultimate Strength	2,745	0.092	
Wave/Current Loads on Deck	API 20th Edition	145	0.809	0.76
	API Section 17 - Design Level	38	0.858	
	API Section 17 - Ultimate Strength	333	0.452	
Total Base Shear	API 20th Edition	2,489	0.072	0.07
	API Section 17 - Design Level	1,657	0.069	
	API Section 17 - Ultimate Strength	3,155	0.072	

Note that in the above table, the wind loads and wave/current loads on the deck have lower mean estimates and larger COVs. For example, for the 20th Edition criteria and for three wave approach directions, the mean wind loads vary from 53 kips to 67 kips, the mean wave/current-in-deck loads vary from 50 kips to 145 kips, whereas the mean wave/current loads on the jacket vary from 1,652 kips to 2,277 kips.

The larger differences in wind and wave/current in deck loads are attributed to differences in the computation of wave crest height and water particle velocities, which are dependent on the participants' individual computer program and assumptions.

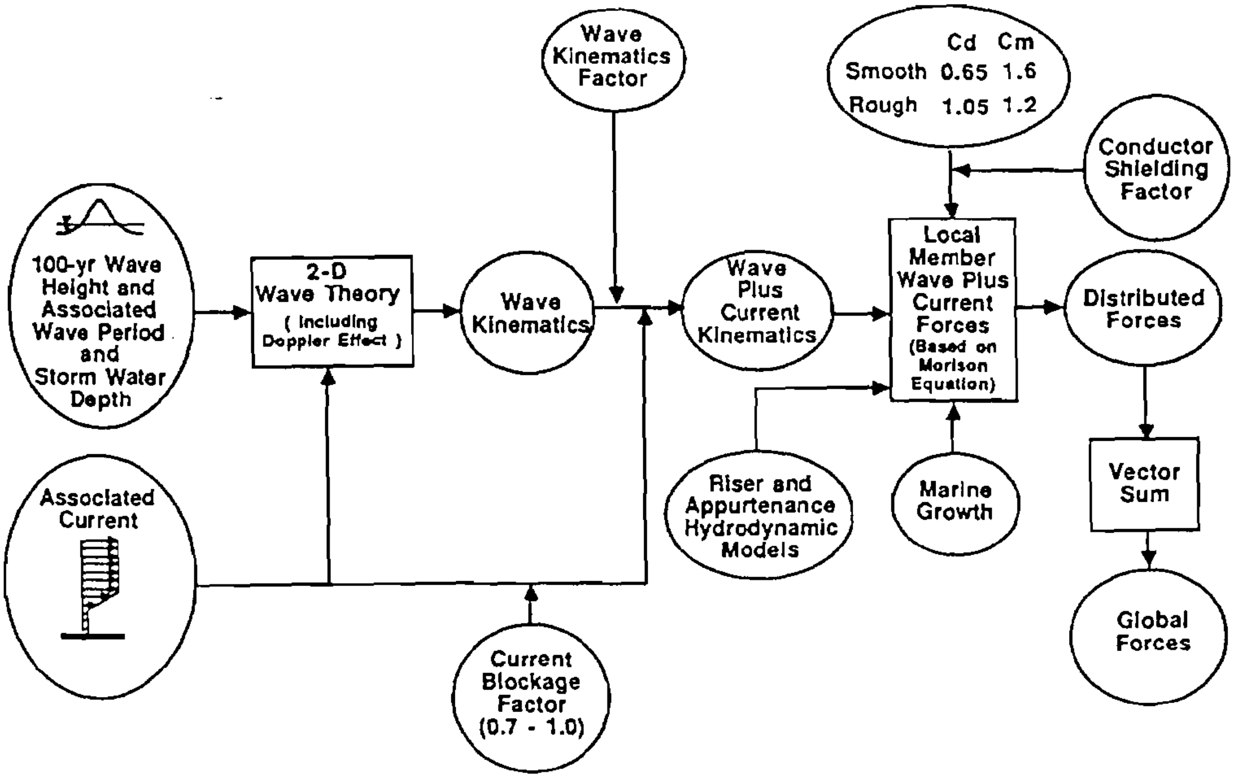
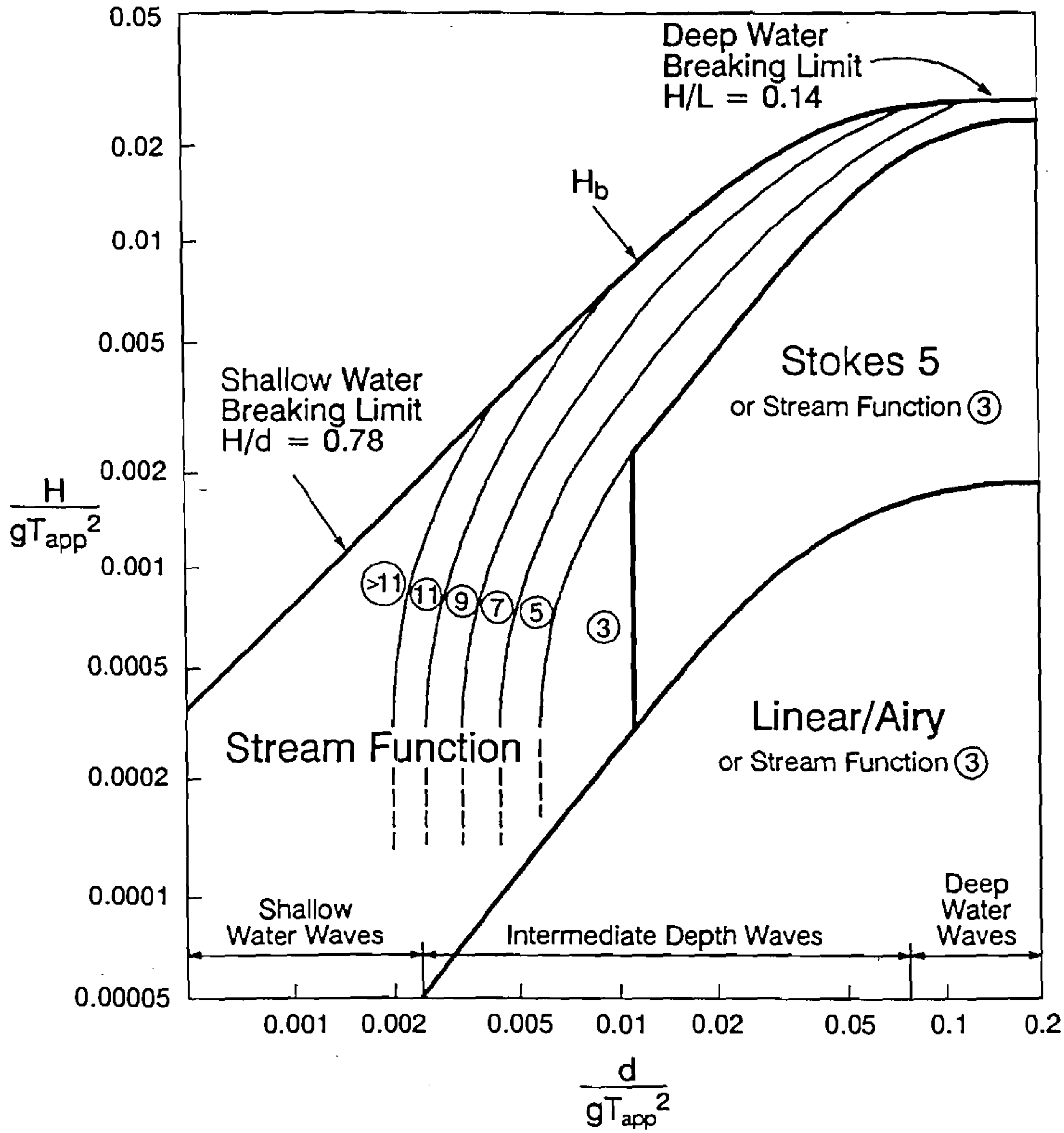


Figure 5-1 Procedure for Calculation of Wave Plus Current Forces for Static Analysis (Refer Fig. 2.3.1-1 of API RP 2A, 20th Ed.)



H/gT_{app}^2: Dimensionless wave steepness
d/gT_{app}^2: Dimensionless relative depth
H: Wave height
H_b: Breaking wave height
d: Mean water depth
T_{app}: Wave period
g: Acceleration of gravity

Figure 5-2 Regions of Applicability of Stream Function, Stokes V, and Linear Wave Theory (Refer Fig. 2.3.1-3 of API RP 2A, 20th Ed.)

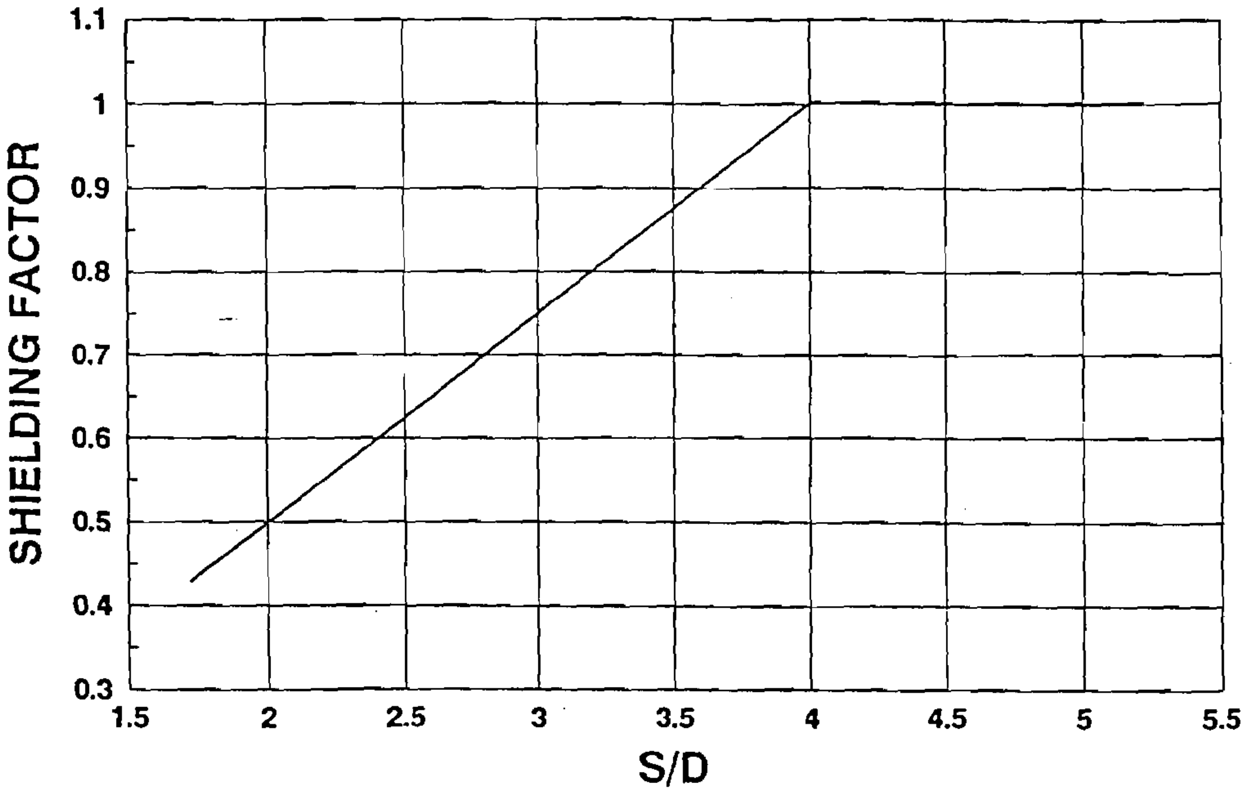


Figure 5-3 Shielding Factor for Wave Loads on Conductor Array as a Function of Conductor Spacing (Refer Fig. 2.3.1-4 of API RP 2A, 20th Ed.)

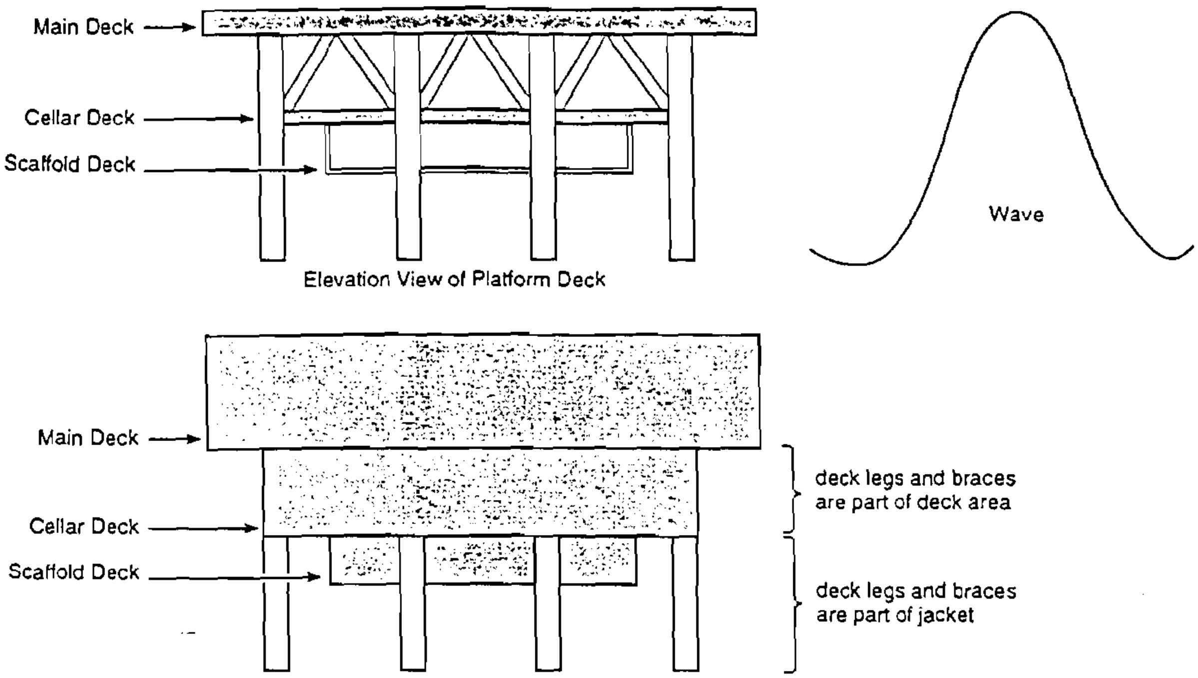


Figure 5-4a Silhouette Area Definition

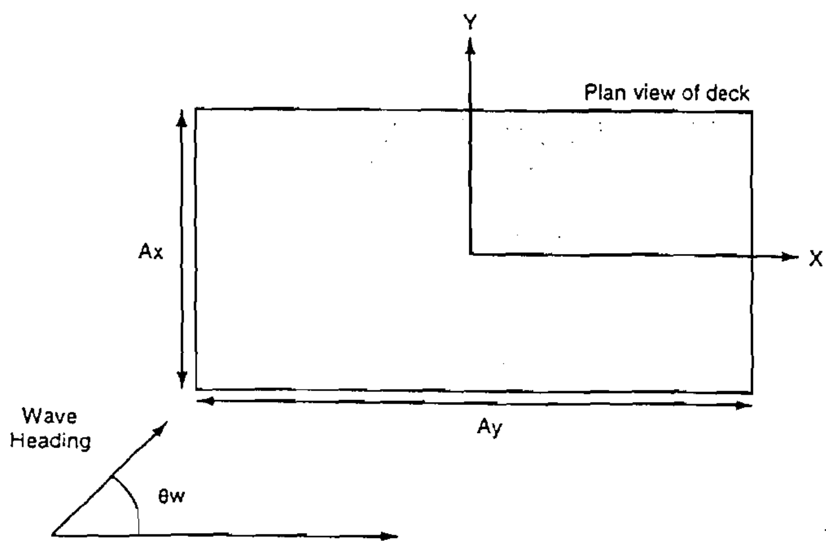


Figure 5-4a Wave Heading and Direction Convention

Figure 5-4 Section 17 Wave-in-Deck Loads Computation Procedure (Refer Fig. C.17.6.2-1b of Supplement 1 API RP 2A, 20th Ed.)

Metocean Criteria		API 20th Edition Force, kips			API Sect 17 - Design Level Force, kips			API Sect 17 - Ultimate St Force, kips			
Direction,degrees		315	270	225	315	270	225	315	270	225	Avg
Wind	A	52	52	47	35	33	31	59	56	53	
	B	58	77	52	38	51	34	65	87	58	
	C	57	78	52	40	53	36	62	84	57	
	D	59	79	53	39	52	35	67	89	59	
	E	59	56	55	41	39	36	67	63	59	
	F	54	72	48	36	48	32	61	82	55	
	G	18	18	16	12	12	10	20	20	18	
	H	90	80	85	65	60	60	110	105	105	
	I	82	74	74	55	52	49	88	84	83	
	J	64	86	58	42	57	38	72	98	66	
	K	50	65	44	33	46	29	58	74	50	
	Mean	58	67	53	40	46	35	66	76	60	
	COV	0.299	0.275	0.312	0.317	0.282	0.331	0.313	0.292	0.337	0.306
Wave & Current on Jacket	A	1,965	2,259	1,730	1,583	1,602	1,519	2,337	2,707	2,045	
	B	1,799	2,094	1,521	1,442	1,452	1,381	2,166	2,531	1,839	
	C	1,797	2,119	1,490	1,413	1,423	1,325	2,129	2,512	1,769	
	D	1,927	2,264	1,698	1,564	1,580	1,563	2,323	2,737	2,049	
	E	1,955	2,271	1,641	1,600	1,593	1,509	2,380	2,778	1,974	
	F	2,173	2,558	1,827	1,766	1,785	1,679	2,625	3,097	2,209	
	G	2,066	2,322	1,737	1,687	1,636	1,607	2,460	2,786	2,085	
	H	2,325	2,770	1,915	1,830	1,830	1,740	2,865	3,360	2,370	
	I	1,806	2,135	1,508	1,448	1,463	1,372	2,174	2,575	1,826	
	J	1,788	2,114	1,541	1,443	1,462	1,407	2,134	2,538	1,859	
	K	1,850	2,144	1,565	1,492	1,484	1,425	2,232	2,579	1,884	
	Mean	1,950	2,277	1,652	1,570	1,574	1,503	2,348	2,745	1,992	
	COV	0.086	0.089	0.081	0.086	0.083	0.085	0.093	0.092	0.088	0.087
Wave & Current on Deck	A	25	38	14	9	9	7	60	166	26	
	B	266	434	154	107	115	103	451	709	284	
	C	85	145	49	35	39	33	159	355	91	
	D	80	131	46	30	33	29	136	338	90	
	E	109	123	40	48	47	41	127	144	96	
	F	86	133	42	31	31	27	153	295	81	
	G	70	170	0	0	0	0	145	314	82	
	H	0	0	0	0	0	0	20	195	0	
	I	85	100	55	35	38	34	165	345	95	
	J	189	286	115	81	81	78	370	482	204	
	K	44	36	33	38	27	34	227	316	37	
	Mean	95	145	50	38	38	35	183	333	99	
	COV	0.754	0.808	0.895	0.823	0.858	0.850	0.657	0.452	0.779	0.764
Total	A	2,042	2,349	1,791	1,626	1,644	1,558	2,456	2,929	2,124	
	B	2,122	2,605	1,726	1,588	1,619	1,518	2,682	3,326	2,181	
	C	1,940	2,342	1,591	1,486	1,515	1,393	2,350	2,951	1,917	
	D	2,066	2,474	1,797	1,633	1,665	1,627	2,525	3,164	2,198	
	E	2,123	2,450	1,736	1,689	1,679	1,586	2,574	2,985	2,129	
	F	2,313	2,764	1,917	1,833	1,864	1,738	2,839	3,474	2,345	
	G	2,154	2,510	1,753	1,699	1,648	1,617	2,625	3,120	2,185	
	H	2,415	2,850	2,000	1,895	1,890	1,800	2,995	3,660	2,475	
	I	1,973	2,309	1,637	1,538	1,553	1,455	2,427	3,004	2,004	
	J	2,041	2,486	1,714	1,563	1,600	1,523	2,576	3,118	2,126	
	K	1,944	2,244	1,642	1,563	1,554	1,489	2,515	2,969	1,971	
	Mean	2,103	2,489	1,755	1,647	1,657	1,573	2,597	3,155	2,150	
	COV	0.068	0.072	0.066	0.072	0.069	0.072	0.068	0.072	0.071	0.070
Original	mean	2,008	2,210	1,735				2,271	2,699	2,001	
Original	cov	0.19	0.27	0.25				0.22	0.23	0.23	0.232

Table 5-1 Summary of JIP Results - Metocean Loads

6.1 Introduction

The API guidelines for ultimate strength analysis of platforms are given in Section 17.7.3 and Commentary C17.7.3 of Section 17 [1]. The platforms classified under different exposure categories that bypass or do not pass the screening and/or design level analyses requirements, must demonstrate adequate strength and stability to survive the Section 17 ultimate strength loading criteria.

The ultimate strength analysis of a platform differs from the analysis used in its original design and the Section 17 design level assessment. The basic differences between the two analyses are:

Design level analysis: The Section 17 design level analysis metocean criteria (such as wave height, current) is lower than the API RP 2A, 20th Edition criteria for a new design. In this analysis, the factors of safety in the material and strength formulations of platform members, joints, and soils are kept similar to the original design. The design level analysis may be sufficient for Section 17 assessment of a platform, when the stresses in all members and joints are within the elastic range using current technology and when the wave inundation does not occur.

Ultimate strength analysis: In the ultimate strength analysis, all known factors of safety are removed from the material and strength formulations of elements, allowing the elements to carry loads up to their individual ultimate strengths. Upon reaching their ultimate strengths the elements may continue to carry the same or reduced loads, depending on their post-ultimate behavior. Some of these overstressed members may exhibit partial or complete failures (such as buckling, yielding, hinging, tearing) and would redistribute loads to other members. The re-distribution of loads would depend upon the framing patterns and variations in loads (or level of capacity utilization) in different members. A platform would meet the Section 17 ultimate strength metocean criteria in its intact or partially damaged state if the highest calculated lateral capacity exceeds the applied loads.

Basic global failure mechanisms are defined below. In many cases, the element failures may occur together in different zones of a platform (such as deck, jacket, pile, and soil) up to the formation of failure mechanisms:

- Deck legs failure: indicated by formation of fully plastic hinges in multiple legs at one or two levels (e.g., at ends of unsupported spans of the deck legs).
- Jacket frame failure: indicated by failure of joints and braces followed by yield/hinge formations in the legs.
- Pile foundation lateral failure: indicated by fully plastic hinge formation in multiple piles at one or two levels (e.g., near the mudline and/or at some depth below the mudline).
- Pile foundation axial failure: pile pullout/plunging failures indicated by mobilization of full resistance of the q-z springs (tip end bearing) and all t-z (axial) springs for piles.

Section 17.7.3 and its Commentary also suggest the use of elastic methods to determine the ultimate strength of platforms failing the design level assessment. Such methods are suggested for linear global analysis and for local overload considerations. These intermediate analyses may be found useful for some platforms which have very little inelastic behavior in their elements at loads corresponding to the ultimate metocean criteria and which have framings with low redundancy (load redistribution behavior). In this way, the effort required to perform detailed nonlinear analysis (global inelastic) can be eliminated for some platforms, since the platform may pass this conservative analysis procedure.

6.2 Ultimate Strength Analysis

The type of analysis performed is typically referred to as "static pushover" and involves defining a representative profile of lateral forces (wind, wave, and current) acting on the platform (including any wave forces acting on the deck) and then applying this profile with incrementally increasing amplification factors until the platform's ultimate strength is defined. The ultimate strength of the platform can then be used to estimate the wave height that would induce platform collapse or it can be compared with the loads due to any reference level loading (e.g., the 100-year return period wave) to determine the platform's reserve strength.

The basic loading profile corresponds to the metocean loads computed in Section 5.6, at a load step (or wave crest position with respect to the platform) with the maximum base shear. The loads at each node of the platform at this stage are determined and are varied with an increasing amplification factor, as the pushover analysis progresses.

The static pushover analysis consists of the following steps:

1) Generate basic pushover load profile and determine loads at platform nodes
2) Perform capacity analysis using a factored load profile (e.g., with nodal loads as 20% of those per the basic load profile)
3) Identify and check members (legs, piles, braces, joints, etc.) for their response (elastic or inelastic range, element strengths) to the element loads at the pushover analysis load step
4) Modify strengths and stiffnesses of the overstressed or failed elements to represent their post-ultimate behavior
5) Repeat the capacity analysis using the modified strengths and stiffnesses of elements
6) Repeat Steps 2 to 5 for lateral loads at the next step
7) Monitor the lateral loads applied to the platform to determine the predictions of lateral loads at which key response states occur (e.g., successive element failures, large displacements)
8) Determine the ultimate strength (peak lateral loading and/or large lateral displacement at the deck level), as the stage at which a failure mechanism develops due to inelastic

behavior (events) in several elements and/or increased loads due to overturning effect from large displacements for the platform.

9) Determine the post-ultimate behavior of the platform - reduction in lateral loads with increasing displacement.

The above procedure provides the more common approach followed to perform ultimate strength analysis of offshore steel jacket platforms. This approach is an improvement over the "member replacement" method, which utilized linear elastic analysis computer programs used in conventional design of platforms. With the availability of sophisticated full-scope nonlinear analysis software, by which the various failure modes and inelastic behaviors of different elements could be explicitly included, a more accurate analysis can be performed. Additional refinements of the nonlinear analysis software have been made by some companies to automate the procedures and to include more complex nonlinear behavior (such as joint).

In a recent JIP [18, 19], more accurate predictions of lateral loads at element failures and of the ultimate strength were made by using variable pushover load profiles (i.e., including the variation in the vertical centroid of the load pattern with the wave height). This method becomes particularly useful for platforms in which the waves inundate the deck.

The benchmark capacity analysis was performed for the following two cases:

- with complete foundation (soil-structure interaction) effect modeled
- with the foundation failure modes (pile/soil events) suppressed by modeling the pileheads fixed at the mudline

The analysis results for these cases determined by one JIP participant are discussed in detail in Sections 6.3 and 6.4 and a comparison of results obtained by all JIP participants is given in Section 6.5.

6.3 Base Case Capacity Analysis Results

The nonlinear model developed in Section 4 was analyzed for the pushover load pattern developed in Section 5 using PMB's CAP computer software [20]. The analysis was performed for storms approaching from 270 and 315 degrees. The results obtained are presented in Figures 6-1 to 6-3 and are discussed below.

Diagonal Direction (270 degrees from True North): Figure 6-1 presents the load vs. displacement plot with the deck displacements as the discriminator. The first event (inelastic event) occurs at a lateral load level of 1,920 kips due to first yield of a section (Figure 6-2). Following this, two piles would plunge or pullout. The ultimate strength is defined as 2,070 kips (Figure 6-3). The curvature in the so called linear portion of the analysis is associated with the non-linear pile-structure interaction.

The inelastic events identified from the analysis are given in the following table:

Load Step	Lateral Displacement at Deck Level (ft)	Pushover Lateral Load (kips)	Elements with Inelastic Events	Type of Component Failure Mode
27	2.86	1,920	Leg B2	First yield of leg section
35	3.50	2,070	Pile B2	First pile pulls out
40	4.70	2,070	Pile A1	Second pile plunges

The load and capacity analyses results are further discussed as follows:

API RP 2A, 20th Edition Reference level load, S_{ref} = 2,340 kips
Section 17 Ultimate load level, S_{ult} = 2,950 kips

Load level at first element failure = 1,920 kips
Ultimate strength, R_u = 2,070 kips
Platform failure mode Foundation failure

Ultimate strength to load at first element failure, R_1 = 1.08
Reserve strength ratio, RSR (= R_u / S_{ref}) = 0.88
Ultimate strength to Section 17 criteria, R_{17} = 0.70

<u>End-On Direction (315 degrees from True North)</u>: The load vs. displacement plot using the deck displacements as the discriminator is presented in Figure 6-4. The first event (inelastic event) occurs due to yielding of a pile section at a lateral loading of 2,100 kips (Figure 6-5). Subsequent inelastic events occur in two other piles and two legs with an additional 200 kips lateral loading (Figure 6-7). At this stage the soil axial capacity was fully mobilized, and pullout and plunging occur at the same lateral loading with increased displacements (Figures 6-8 to 6-10).

The inelastic events identified from the analysis are given in the following table:

Load Step	Lateral Displacement at Deck Level (ft)	Pushover Lateral Load (kips)	Elements with Inelastic Events	Type of Component Failure Mode
12	2.57	2,100	Pile A2	First yield in a pile section
14	2.70	2,180	Pile A1	First yield in a pile section
19	2.92	2,255	Leg A2	First yield in a leg section
21	3.02	2,285	Pile A2	First yield in a pile section
			Pile B2	First yield in a pile section
23	3.17	2,300	Leg A1	First yield in a leg section
			Pile A1	First yield in a pile section

The load and capacity analyses results are:

API RP 2A, 20th Edition Reference level load, S_{ref} = 1,940 kips
Section 17 Ultimate load Level, S_{ult} = 2,350 kips

Load level at first element failure = 2,100 kips
Ultimate strength, R_u = 2,300 kips
Platform failure mode Foundation failure

Ultimate strength to load at first element failure, R_I = 1.09
Reserve strength ratio, RSR (= R_u / S_{ref}) = 1.19
Ultimate strength to Section 17 criteria, R_{17} = 0.98

6.4 Fixed Base Capacity Analysis Results

The platform was analyzed using the model with suppression of the platform foundation behavior to obtain results indicative of the jacket structural capacity.

Diagonal Direction (270 degrees from True North): The lateral load vs. deck displacement plot is given in Figure 6-11. Initial events would occur due to first yielding of sections in Legs A-1 and B-2 (Figures 6-12 and 6-13). Thereafter, one K-brace in the third bay (from mudline) would buckle at a lateral loading of 4,870 kips (Figure 6-14), which also defines the ultimate strength of the platform. Upon buckling of this brace, it is predicted that several K-braces in four bays would also buckle and sections in several horizontal braces would yield or become plastic (Figures 6-15 to 6-17) with increased displacements at the deck level. The residual capacity in this case is predicted to be about 3,280 kips.

The inelastic events identified from the analysis are:

Load Step	Lateral Displacement at Deck Level (ft)	Pushover Lateral Load (kips)	Elements with Inelastic Events	Type of Component Failure Mode
4	0.71	3,500	Leg A-1	First yield of a leg section
37	0.83	4,045	Leg B-2	First yield of a leg section
44	0.86	4,145	Leg A-1	First yield of a leg section
			Leg B-2	First yield of a leg section
108	1.08	**4,870**	Vertical frame diag. brace	Compression buckling
121	1.03	4,410	Vertical frame diag. brace	Compression buckling
130	1.03	4,125	Horizontal frame brace	First yield of a horizontal brace section
134	1.06	4,255	Vertical frame diag. brace	Compression buckling
148	1.03	3,830	Vertical frame diag. brace	Compression buckling
158	0.96	3,280	Horizontal frame brace	First yield of a horizontal brace section

The load and capacity analyses results are:

API RP 2A, 20th Edition Reference level load, S_{ref}	= 2,340 kips
Section 17 Ultimate load level, S_{ult}	= 2,950 kips
Load level at first element failure	= 3,500 kips
Ultimate strength, R_u	= 4,870 kips
Platform failure mode	Buckling of braces
Ultimate strength to load at first element failure, R_1	= 1.39
Reserve strength ratio, RSR (= R_u / S_{ref})	= 2.08
Ultimate strength to Section 17 criteria, R_{17}	= 1.65

End-On Direction (315 degrees from True North): The lateral load vs. deck displacement plot is given in Figure 6-18. The first event occurs due to buckling of a brace in the second bay from the mudline at a lateral loading of 3,465 kips (Figure 6-19). Due to this event, buckling of K-braces in six other bays of the platform would occur and leg sections would yield or become fully plastic forming a mechanism (Figures 6-20 to 6-24). At Stage 5, the residual capacity of the platform is predicted as 3,060 kips (Figure 6-23), which would be higher than the loads per Section 17 criteria.

The inelastic events identified from the analysis are given in the following table:

Load Step	Lateral Displacement at Deck Level (ft)	Pushover Lateral Load (kips)	Elements with Inelastic Events	Type of Component Failure Mode
9	0.63	**3,465**	Vertical frame diag. brace	Compression buckling
20	0.58	3,120	Vertical frame diag. brace	Compression buckling
38	0.57	2,965	Vertical frame diag. brace	Compression buckling
54	0.57	2,690	Vertical frame diag. brace	Compression buckling
79	0.69	2,700	Vertical frame diag. brace	Compression buckling
94	0.66	2,475	Vertical frame diag. brace	Compression buckling
111	0.93	2,755	Leg B-1	First yield of a leg section
114	0.95	2,775	Vertical frame diag. brace	Compression buckling
126	0.89	2,470	Vertical frame diag. brace	Compression buckling
144	1.13	2,730	Leg B-2	First yield of a leg section

The load and capacity analyses results are:

API RP 2A, 20th Edition Reference level load, S_{ref}	= 1,940 kips
Section 17 Ultimate load level, S_{ult}	= 2,350 kips
Load level at first element failure	= 3,465 kips
Ultimate strength, R_u	= 3,465 kips
Platform failure mode	Buckling of braces
Ultimate strength to load at first element failure, R_1	= 1.0
Reserve strength ratio, RSR (= R_u / S_{ref})	= 1.79
Ultimate strength to Section 17 criteria, R_{17}	= 1.47

6.5 Variations in JIP Participants' Results

The capacity analysis results submitted by the participants in the JIP varied significantly [4, 5]. A majority of participants analyzed the platform for three storm approach directions. The COV in the estimates of ultimate strength was 23%.

Base Case: The Base Case results from the JIP participants for three directions, with complete soil-structure interaction included, are summarized as follows:

Direction	Parameters	Range of Values	Mean Value	COV
225°	Load at first member with non-linear event -- all 13 participants -- eliminating outliers (J)	 1,200 to 3,530 k 1,200 to 2,290 k	 1,920 k	 0.39
	Ultimate strength -- all 13 participants -- eliminating outliers (J)	 1,610 to 3,570 k 1,610 to 2,830 k	 2,590 k	 0.24
	Reserve Strength Ratio, RSR	0.74 to 1.69	1.32	0.24
270°	Load at first member with non-linear event -- all 13 participants -- eliminating outliers (J)	 980 to 2,295 k 980 to 1,990 k	 1,640 k	 0.23
	Ultimate strength -- all 13 participants -- eliminating outliers (J)	 1,500 to 3,140 k 1,500 to 2,630 k	 2,110 k	 0.23
	Reserve Strength Ratio, RSR	0.57 to 1.13	0.88	0.20
315°	Load at first member with non-linear event -- all 13 participants -- eliminating outliers (J)	 1,060 to 3,420 k 1,060 to 2,435 k	 1,870 k	 0.37
	Ultimate strength -- all 13 participants -- eliminating outliers (J)	 1,550 to 3,440 k 1,550 to 2,895 k	 2,400 k	 0.22
	Reserve Strength Ratio, RSR	0.67 to 1.51	1.16	0.24

Comparisons of load vs. displacement results are given in Figures 6-25 to 6-27. The mean estimates of the ultimate capacities and RSRs for the approach directions 270° and 315° are closer to the estimates given in Section 6.3, whereas the mean estimate for the lateral load level that induces the first inelastic event differs from the estimates given in Section 6.3 for both directions.

However, for these parameters, the range of values among participants' results are significant, with the highest estimates being two to three times the lowest estimates. Somewhat narrower ranges of results are obtained when the results by one participant (J) are not considered.

A majority of participants (ten) indicated that inadequate soil axial compression capacity governed the failure mode in the diagonal direction. Two participants determined platform failure modes due to yielding of pile sections, and one participant found that the failure of the jacket legs and K-braces governed the ultimate strength.

Fixed Base Case: A comparison of load vs. displacement results is given in Figures 6-28 to 6-30. The JIP results for three directions for the Fixed Base Case are summarized as follows:

Direction	Parameters	Range of Values	Mean Value	COV
225° (five participants)	Load at first member with non-linear event	2,010 to 4,200 k	3,240 k	0.25
	Ultimate strength	3,270 to 4,200 k	3,730 k	0.10
	Reserve Strength Ratio, RSR	1.79 to 2.64	2.25	0.16
270° (six participants)	Load at first member with non-linear event	1,100 to 4,060 k	2,780 k	0.40
	Ultimate strength	2,850 to 4,870 k	4,090 k	0.20
	Reserve Strength Ratio, RSR	1.42 to 2.21	1.87	0.19
315° (four participants)	Load at first member with non-linear event	2,210 to 3,910 k	3,230 k	0.22
	Ultimate strength	3,370 to 4,050 k	3,680 k	0.08
	Reserve Strength Ratio, RSR	1.73 to 2.04	1.85	0.09

The above results indicate very low COV for orthogonal directions compared to those for the diagonal (270 degree) direction. The mean estimates for the JIP results for the fixed base case are significantly different than the results presented in Section 6.4 for 270° and 315° directions.

Additional discussion on the sources of variation in the results is provided in "Modifications to and Applications of the Guidelines for Assessment of Existing Platforms Contained in Section 17.0 of API RP 2A." *Proceedings, 27th Offshore Technology Conference, OTC No. 7779* [5]. This paper also provides guidelines regarding acceptable variations in analysis results.

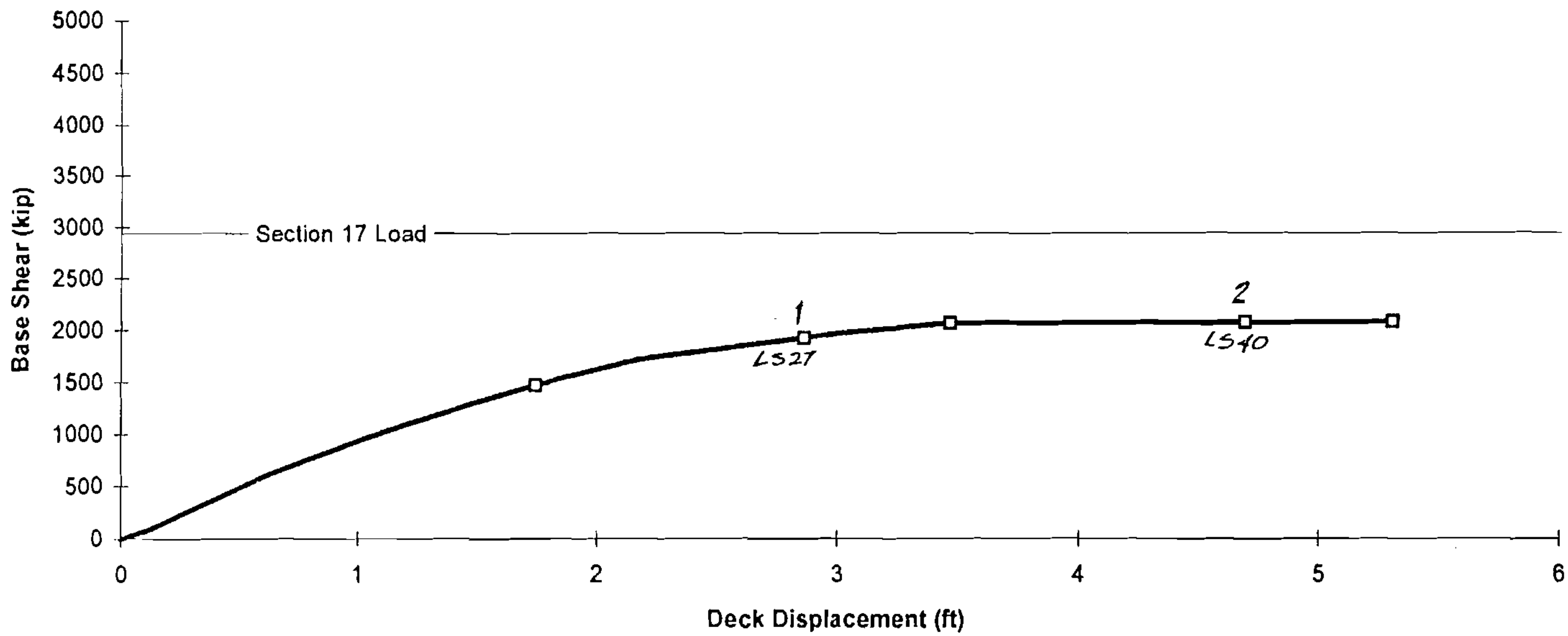
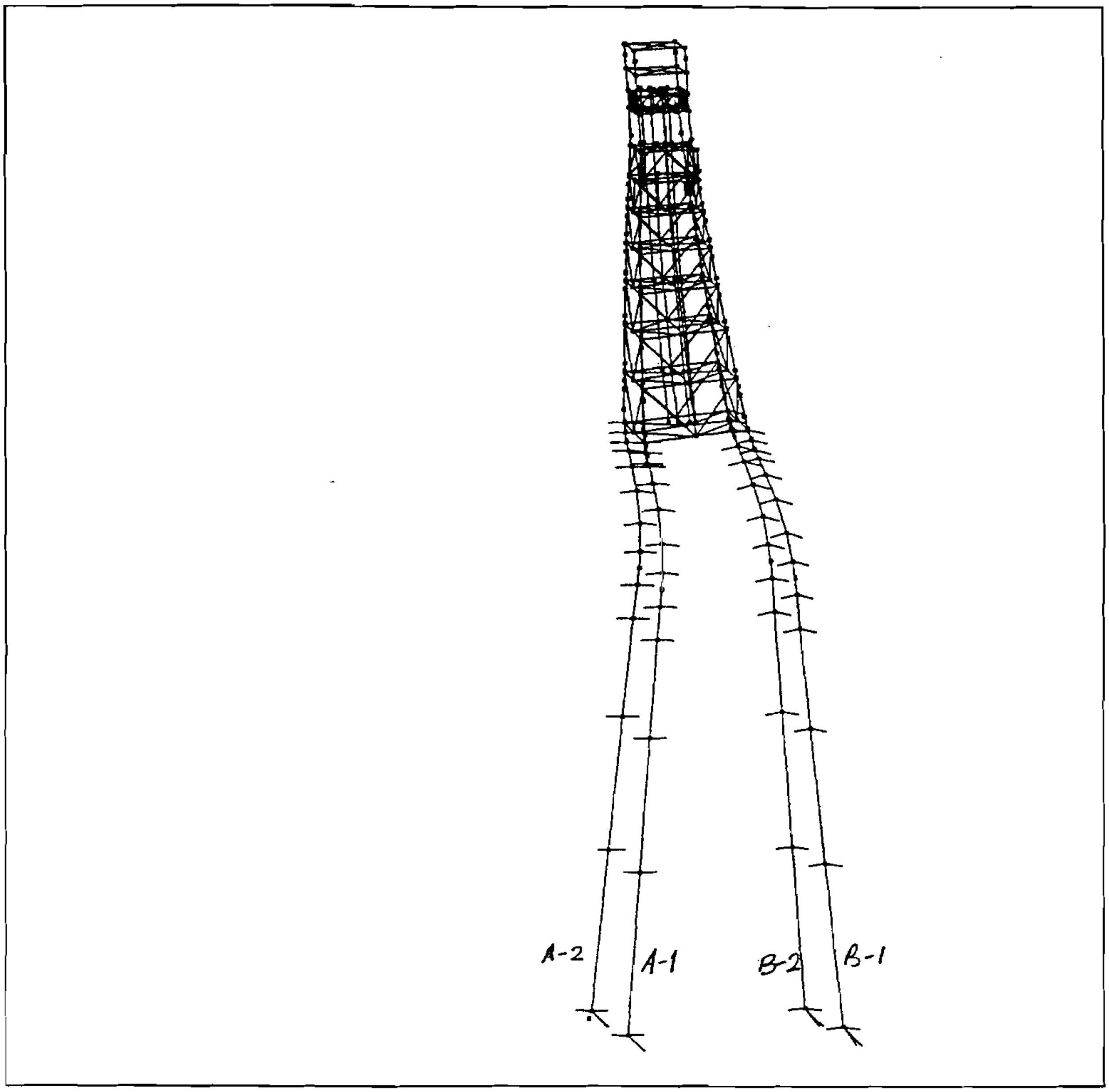


Figure 6-1 **Base Case Load - Displacement Behavior — Diagonal Storm**



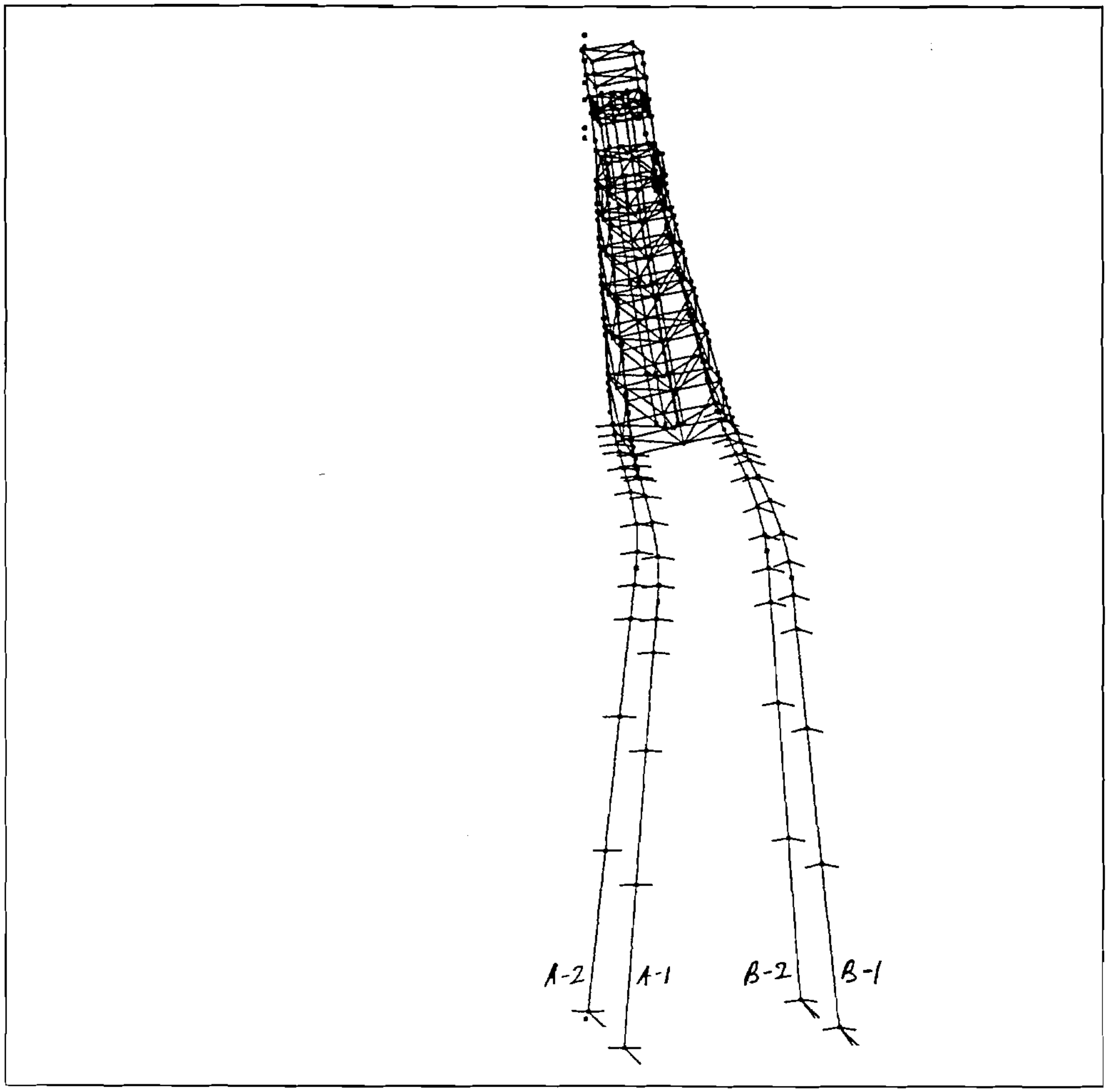
CAP z x

At Load Step 27, Lateral load = 1,920 kips

Inelastic Events Legend

————	Elastic	....	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	....	Fracture

Figure 6-2 Inelastic Events at Stage 1 (per Figure 6-1) — Diagonal Storm



CAP z x

At Load Step 40, Lateral load = 2,070 kips

Inelastic Events Legend

________	Elastic	....	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
________	Beam Clmn Fully Plastic	....	Fracture

Figure 6-3 Inelastic Events at Stage 2 (per Figure 6-1) — Diagonal Storm

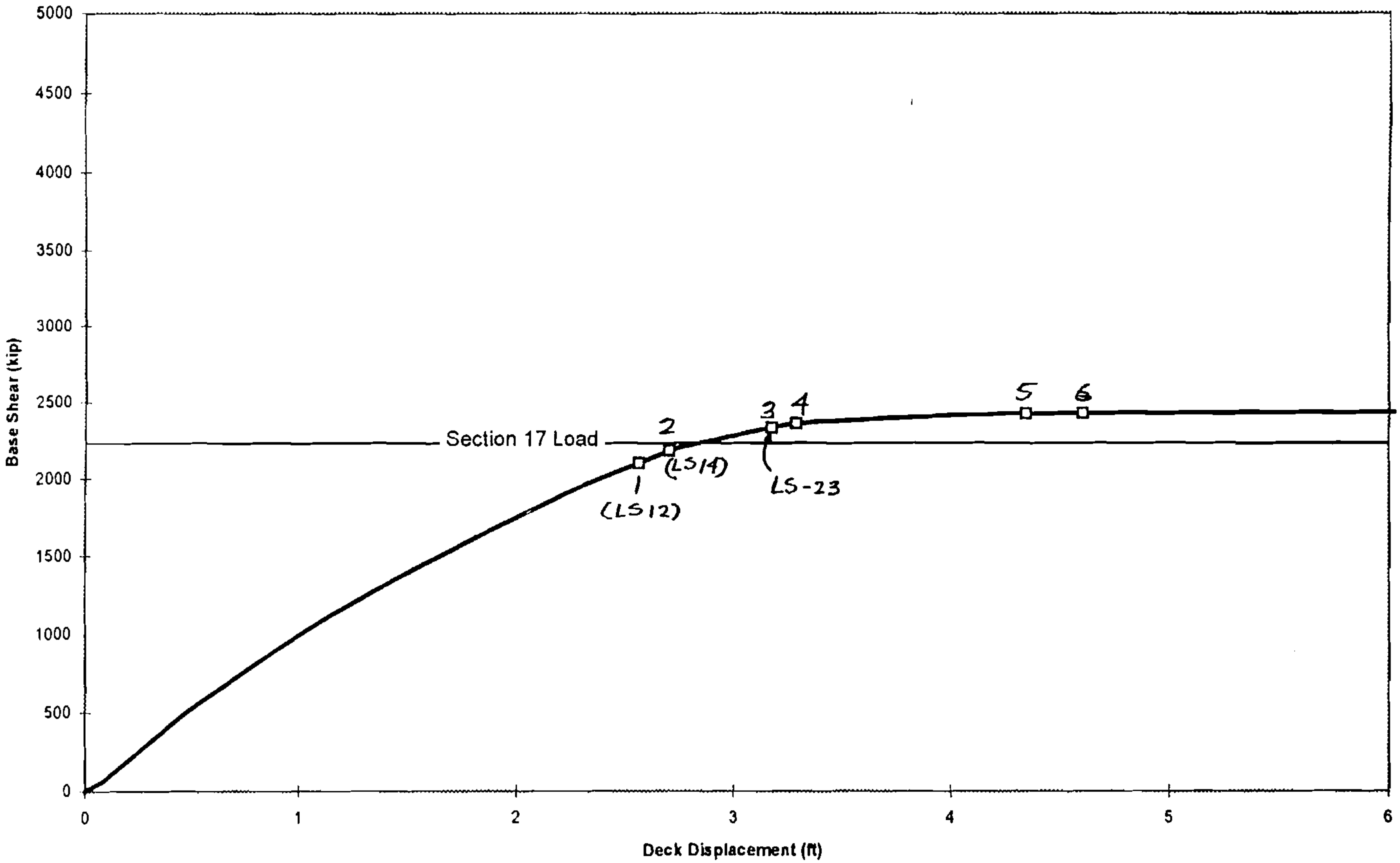
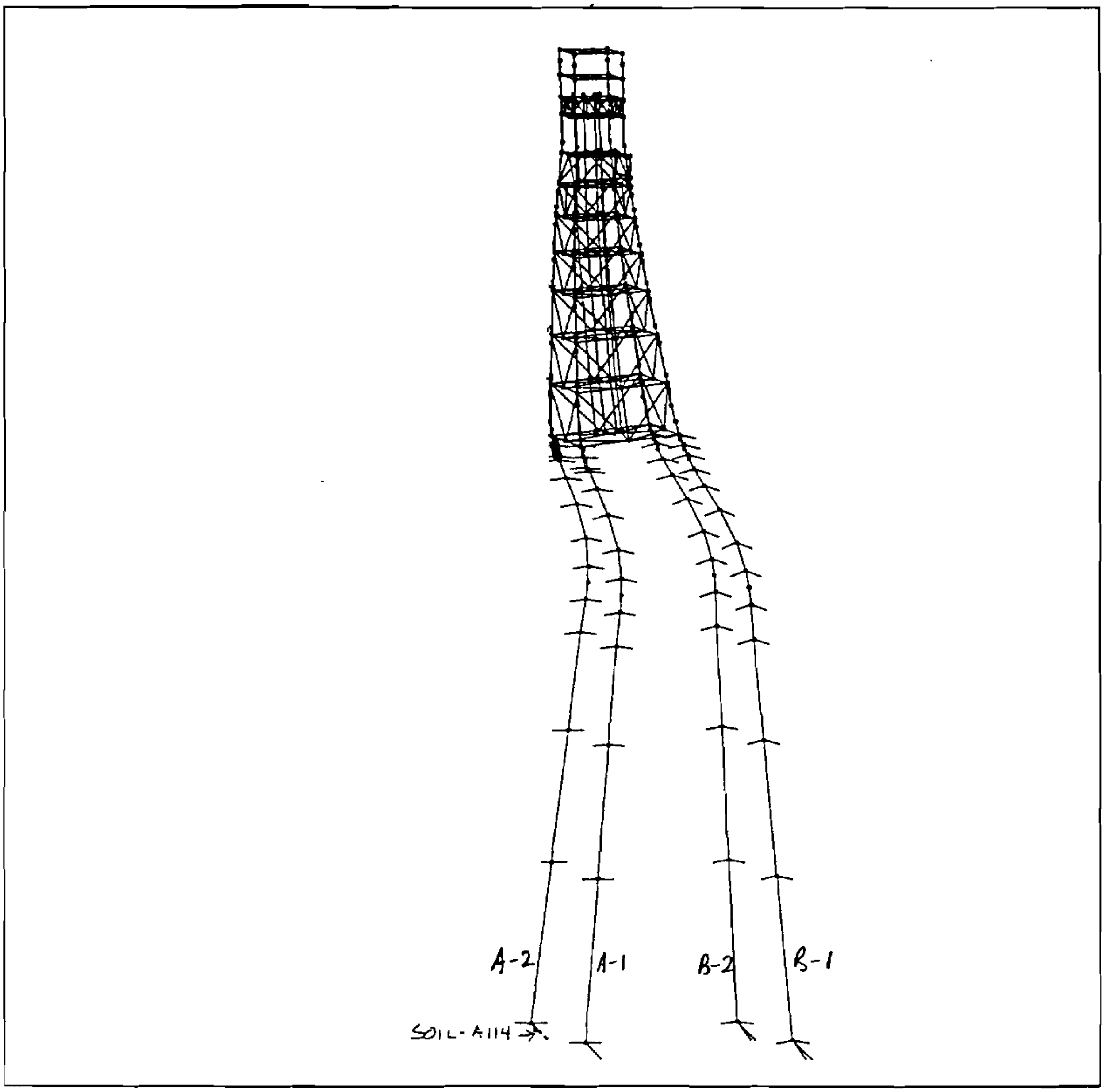


Figure 6-4 Base Case Load - Displacement Behavior — Broadside Storm



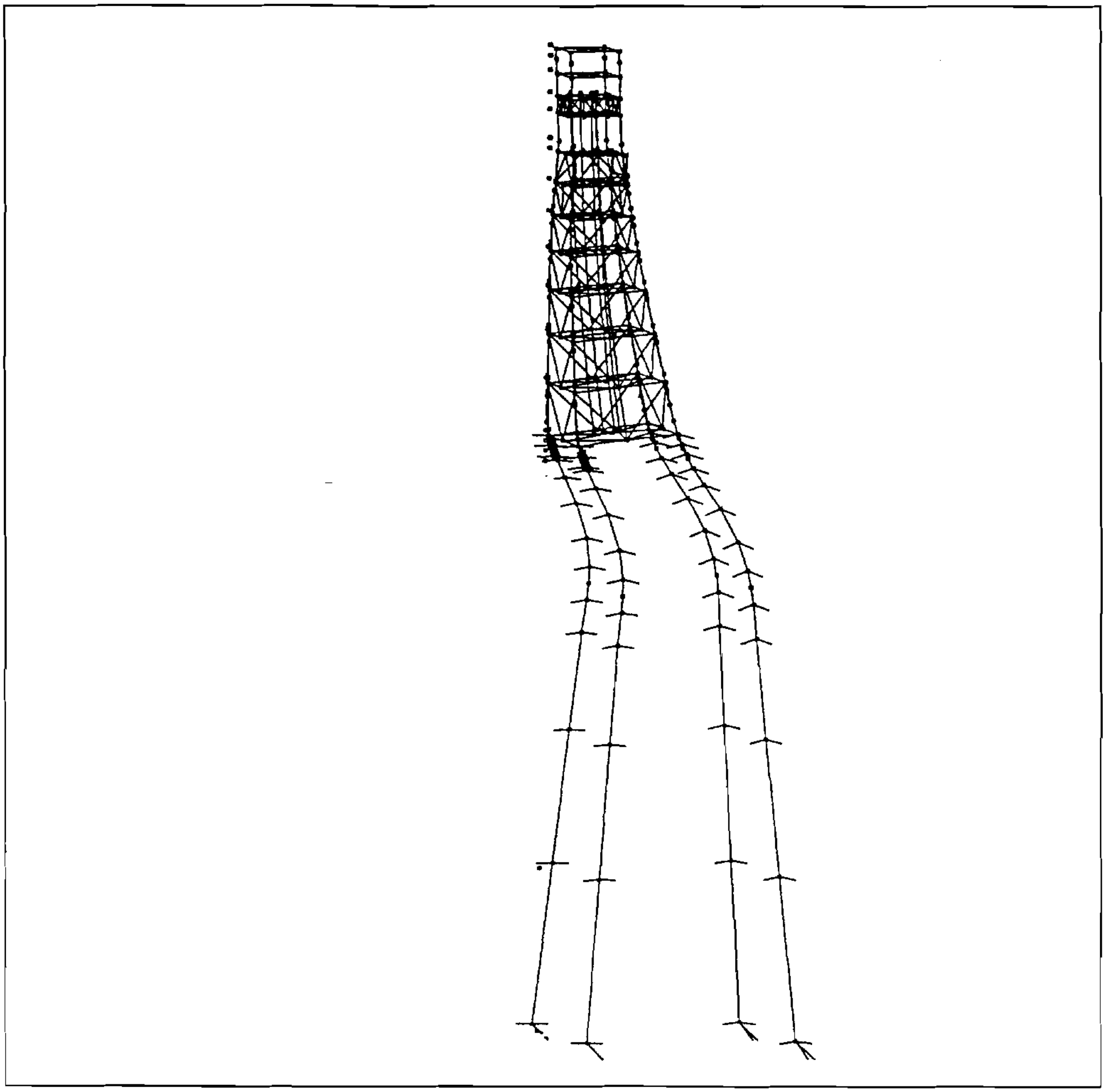
CAP

At Load Step 12, Lateral load = 2,100 kips

Inelastic Events Legend

________	Elastic	________	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
________	Beam Clmn Fully Plastic	....	Fracture

Figure 6-5 Inelastic Events at Stage 1 (per Figure 6-4) — Broadside Storm

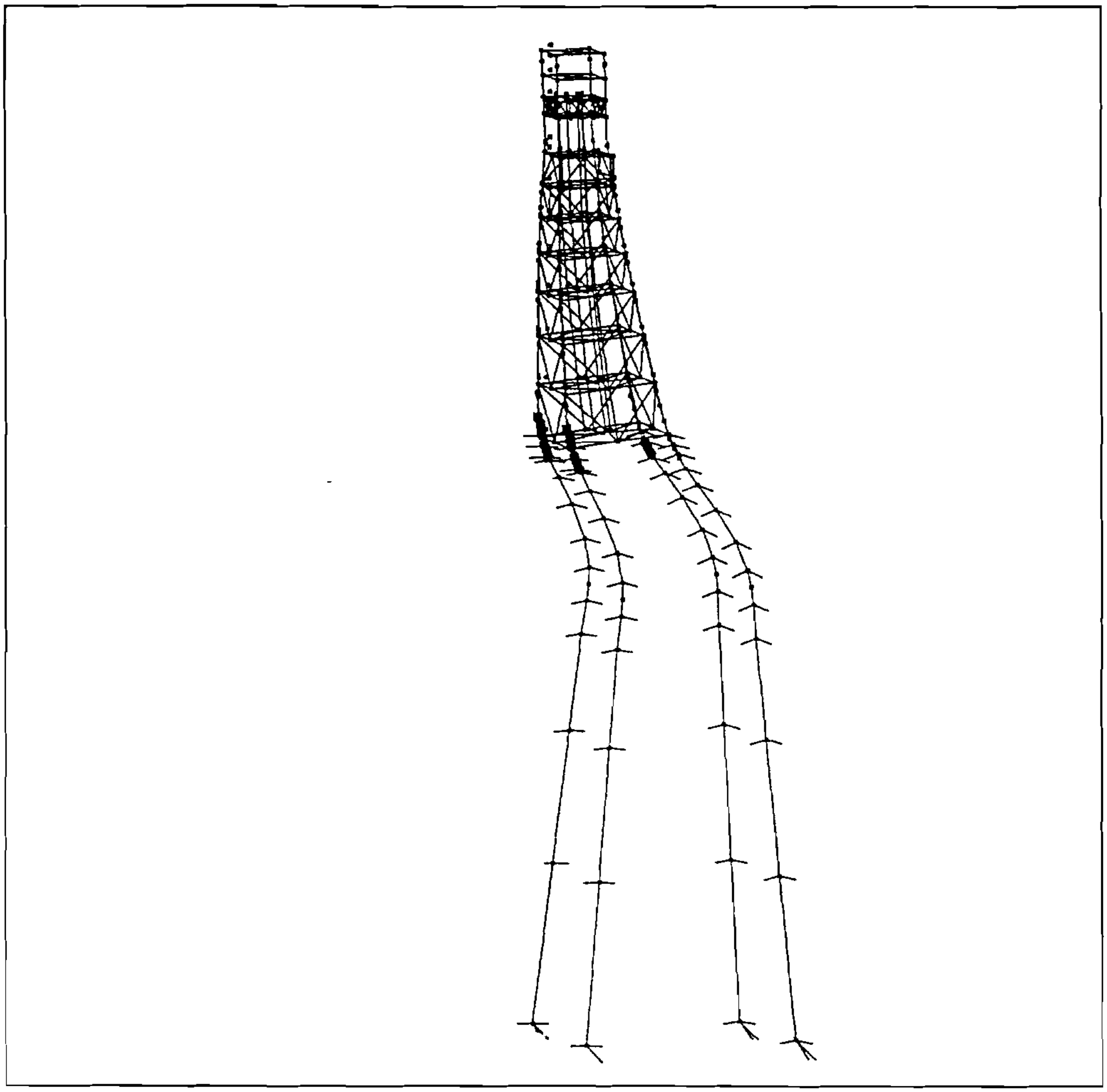


At Load Step 14, Lateral load = 2,180 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-6 Inelastic Events at Stage 2 (per Figure 6-4) — Broadside Storm

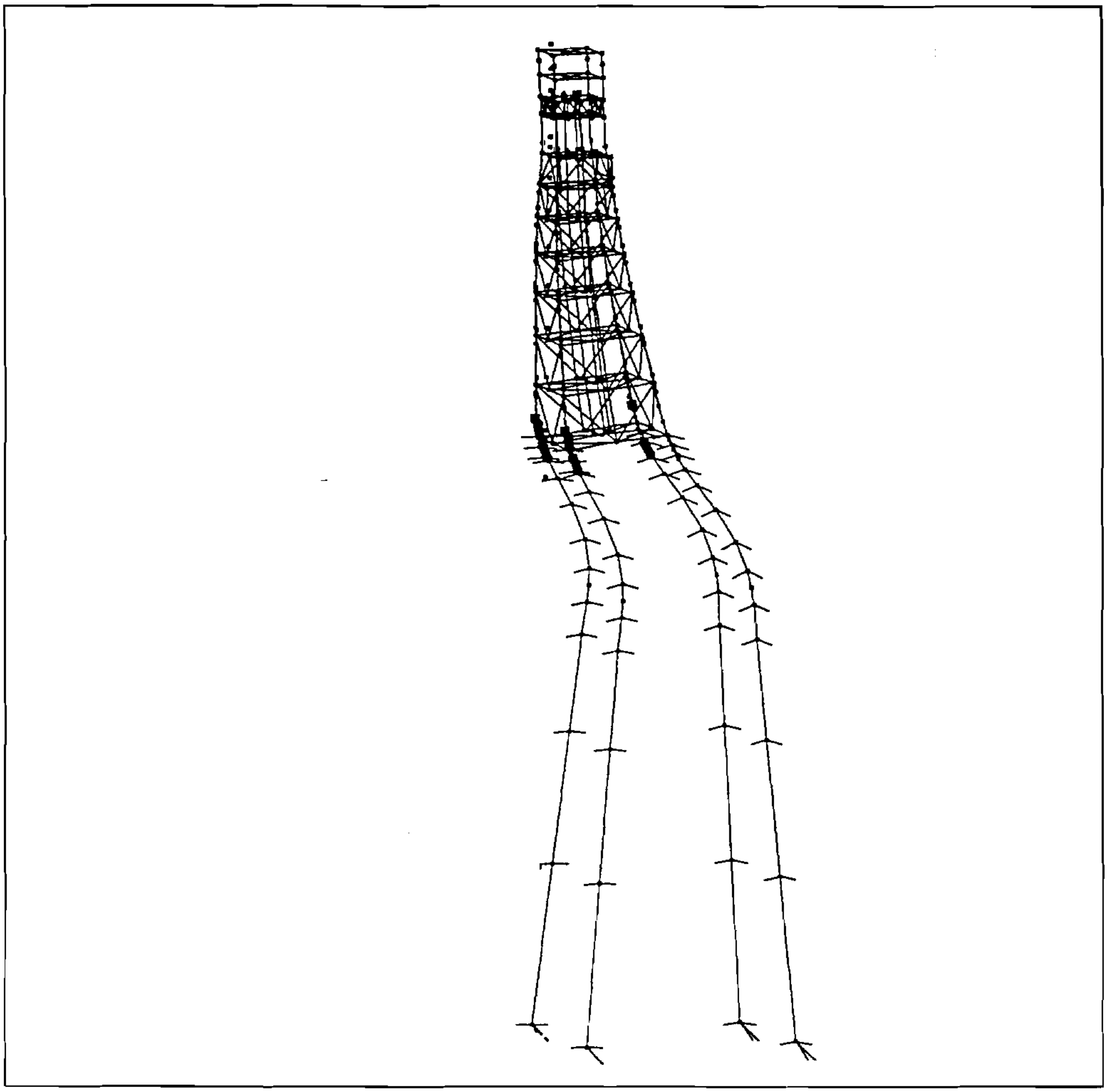


CAP z x — At Load Step 23, Lateral load = 2,330 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-7 Inelastic Events at Stage 3 (per Figure 6-4) — Broadside Storm



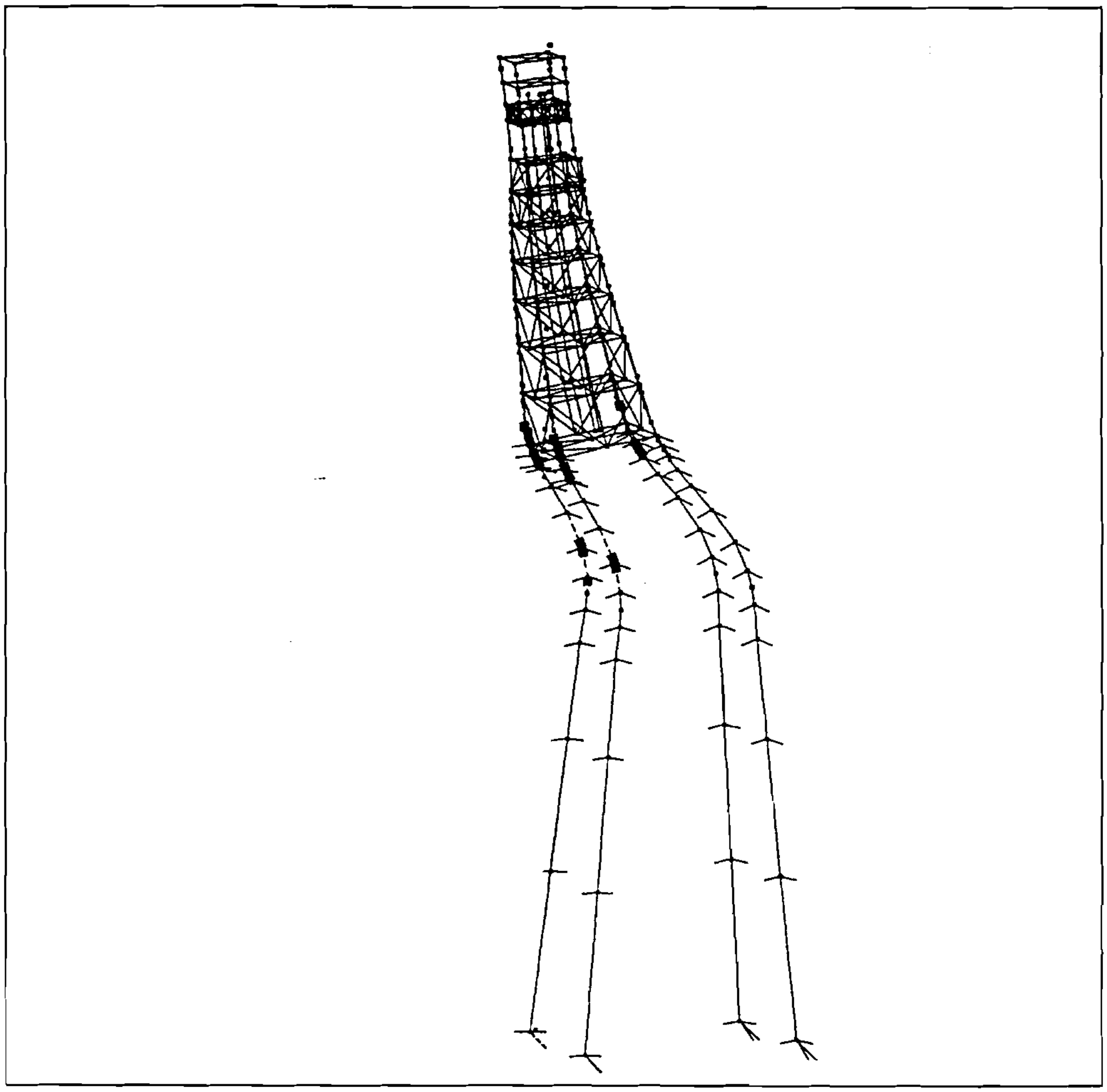
CAP z x

At Lateral load = 2,300 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-8 Inelastic Events at Stage 4 (per Figure 6-4) — Broadside Storm



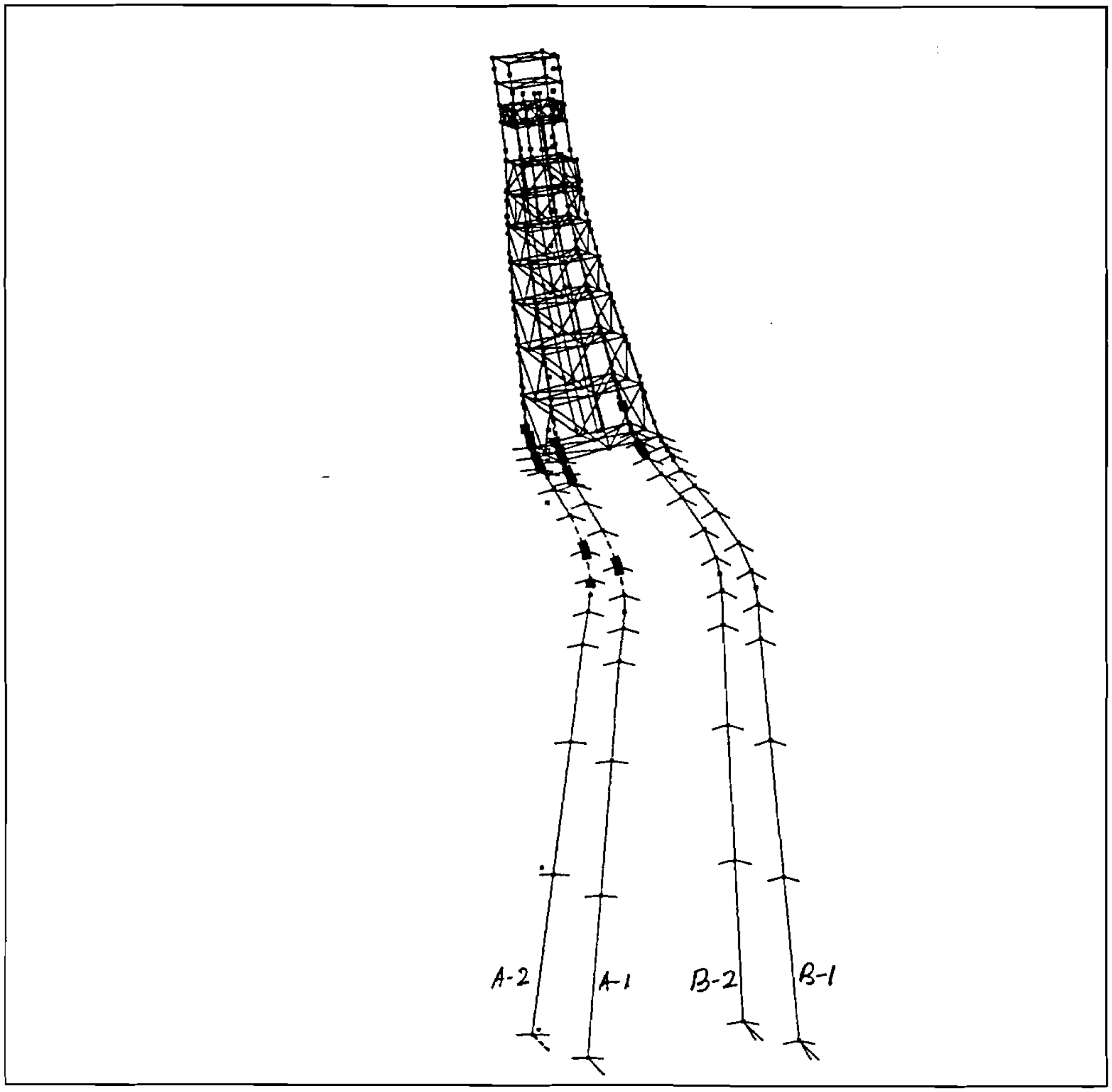
CAP z x

At Lateral load = 2,300 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-9 Inelastic Events at Stage 5 (per Figure 6-4) — Broadside Storm



CAP

At Lateral load = 2,300 kips

Inelastic Events Legend

————	Elastic	————	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	....	Fracture

Figure 6-10 Inelastic Events at Stage 6 (per Figure 6-4) — Broadside Storm

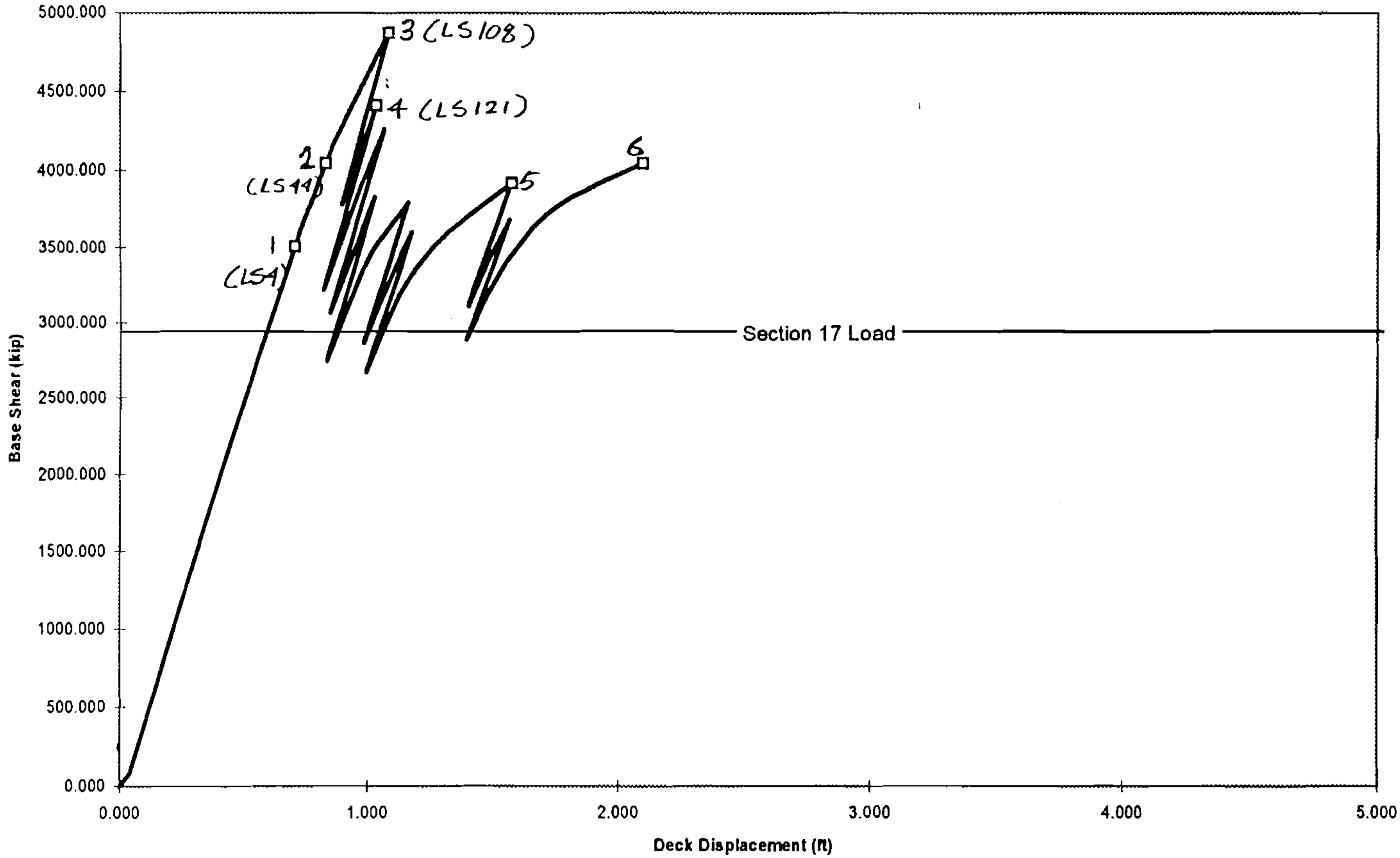
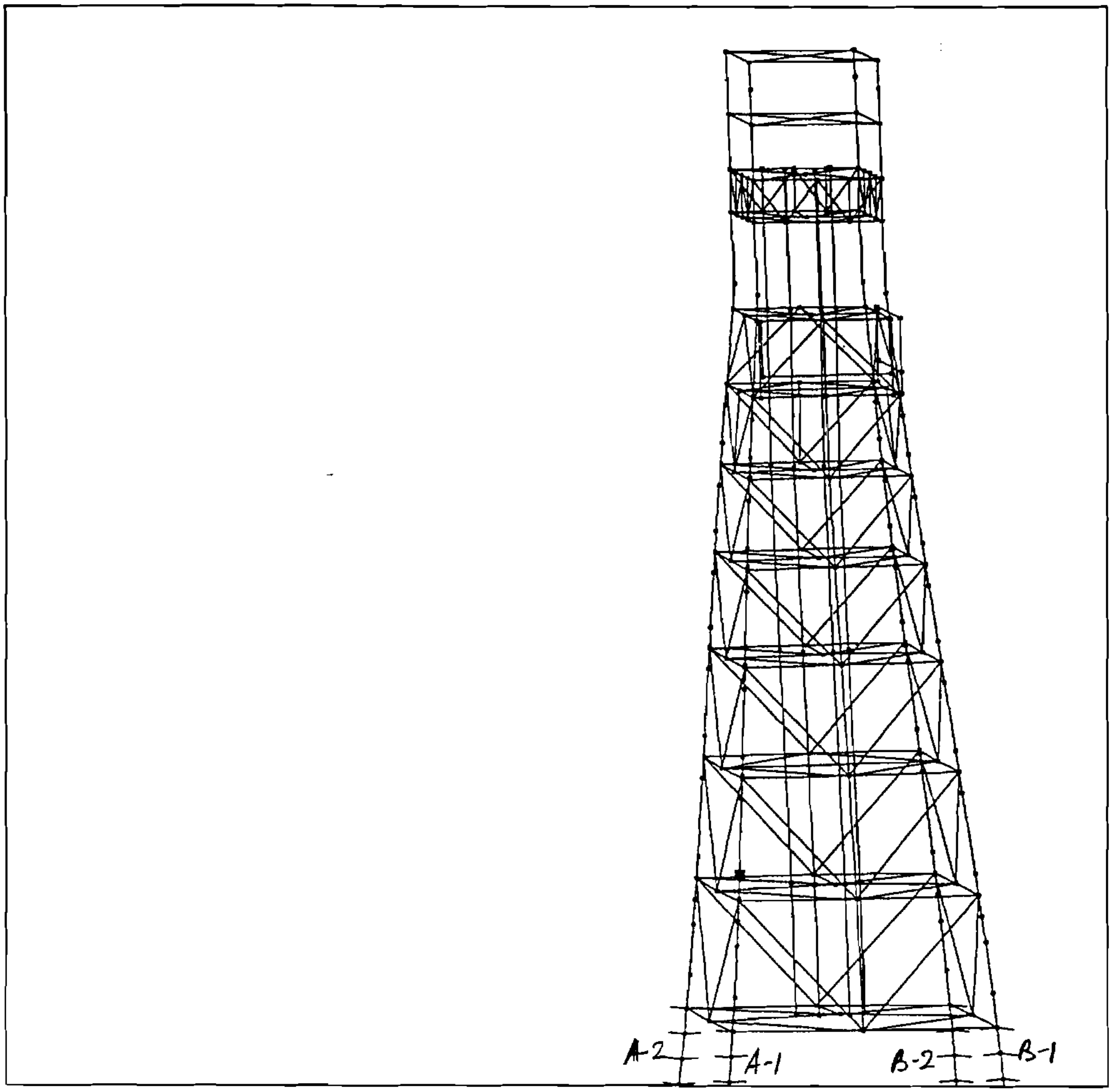


Figure 6-11 Fixed Base Case Load - Displacement Behavior — Diagonal Storm



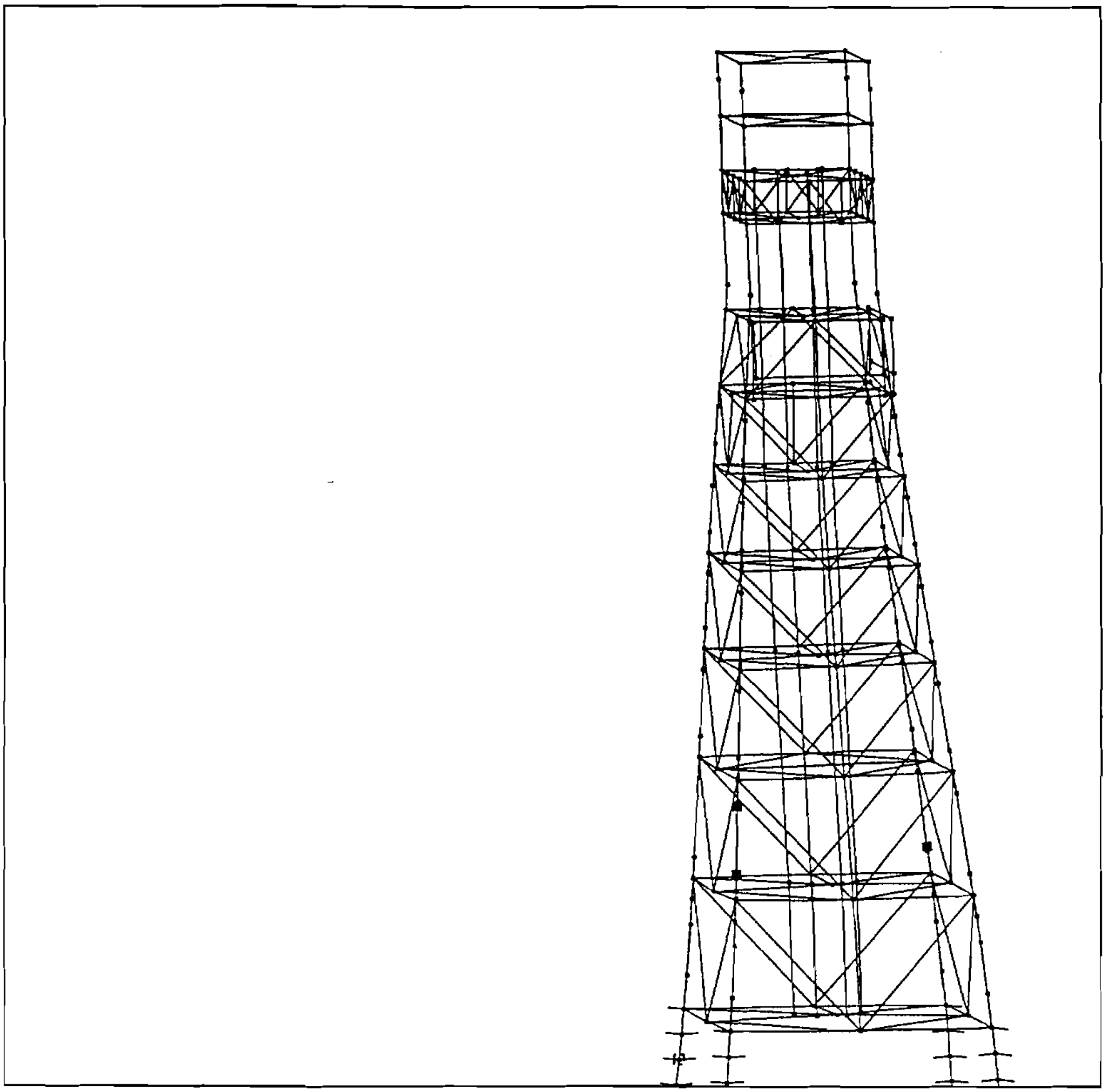
CAP

At Load Step 4, Lateral load = 3,500 kips

Inelastic Events Legend

———	Elastic	———	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
———	Beam Clmn Fully Plastic	....	Fracture

Figure 6-12 Inelastic Events at Stage 1 (per Figure 6-11) — Diagonal Storm

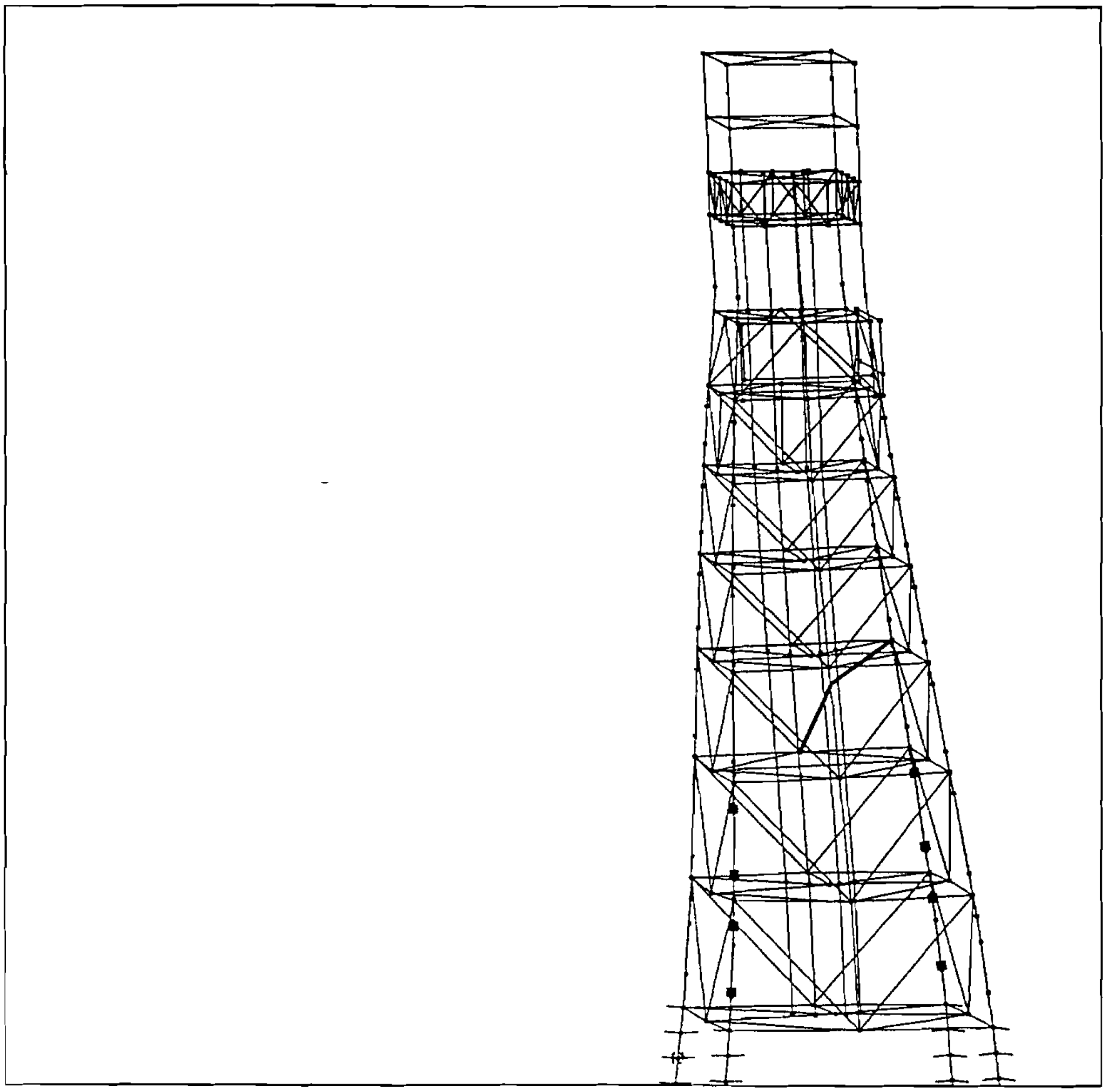


CAP z x At Load Step 44, Lateral load = 4,145 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-13 Inelastic Events at Stage 2 (per Figure 6-11) — Diagonal Storm

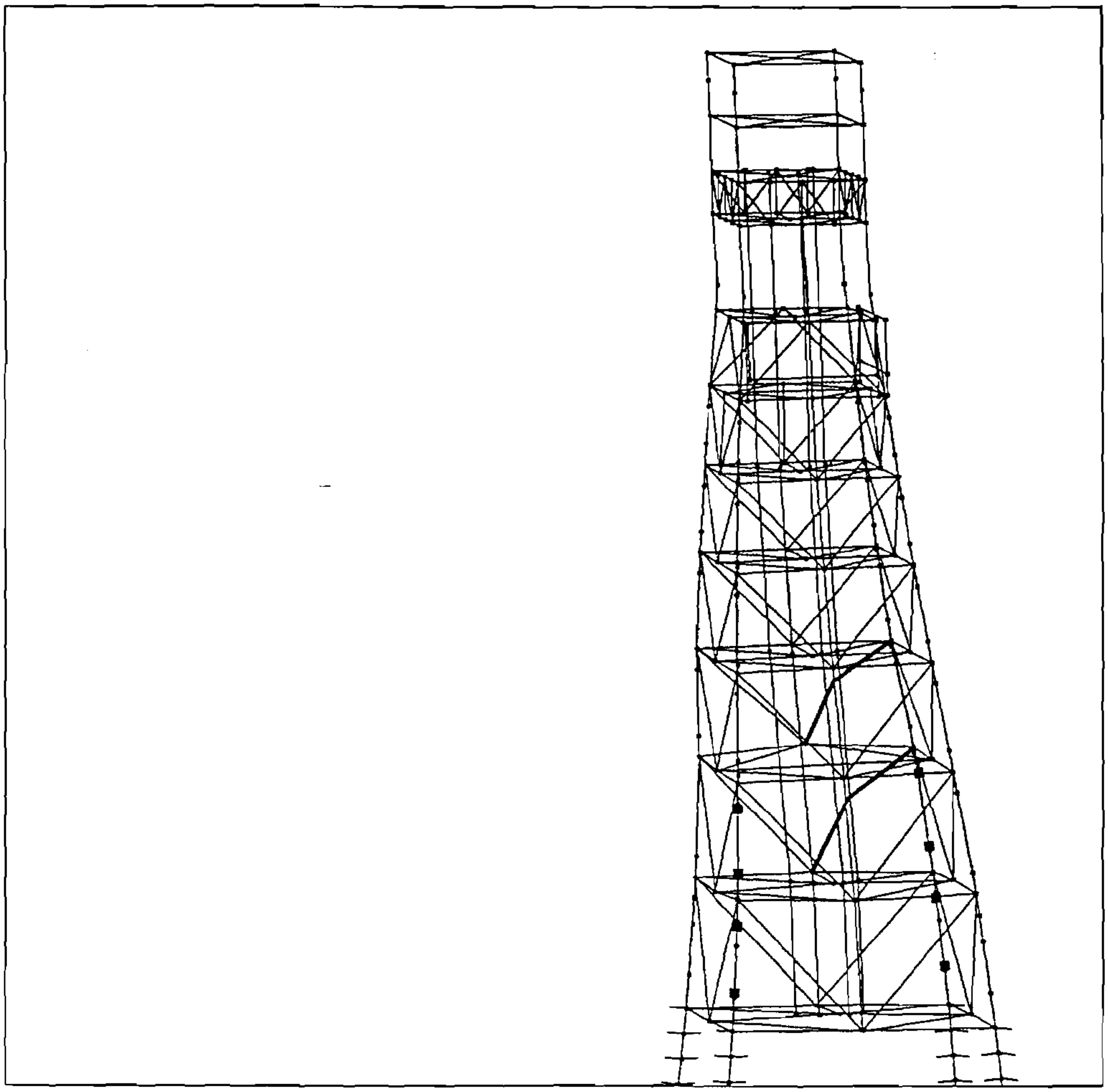


CAP z x At Load Step 108, Lateral load = 4,870 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-14 Inelastic Events at Stage 3 (per Figure 6-11) — Diagonal Storm



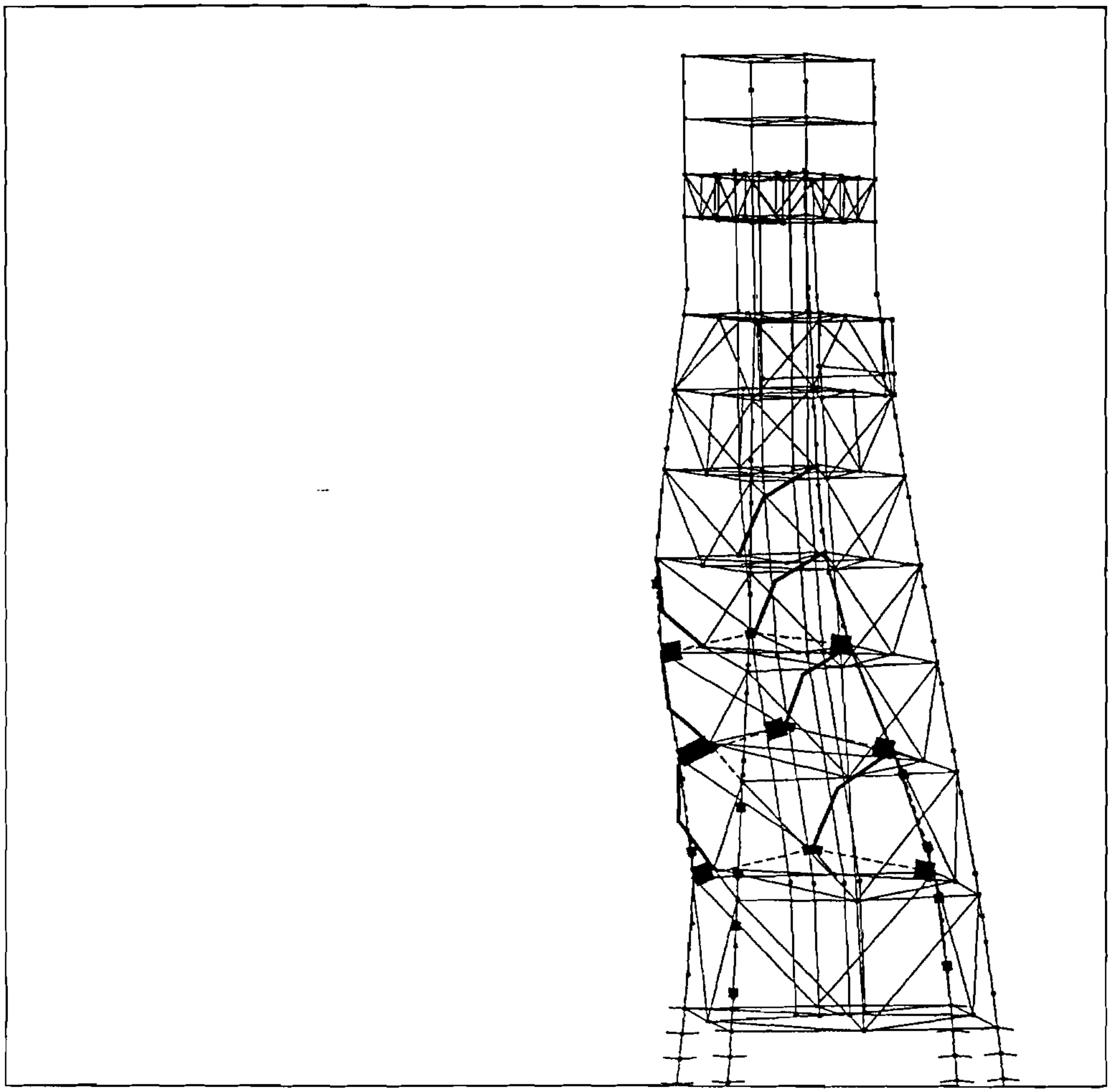
CAP z x

At Load Step 121, Lateral load = 4,410 kips

Inelastic Events Legend

———	Elastic	━━━	Strut Buckling
- - - - -	Strut Residual	-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	- - - - -	Beam Clmn Initial Yield
———	Beam Clmn Fully Plastic	....	Fracture

Figure 6-15 Inelastic Events at Stage 4 (per Figure 6-11) — Diagonal Storm

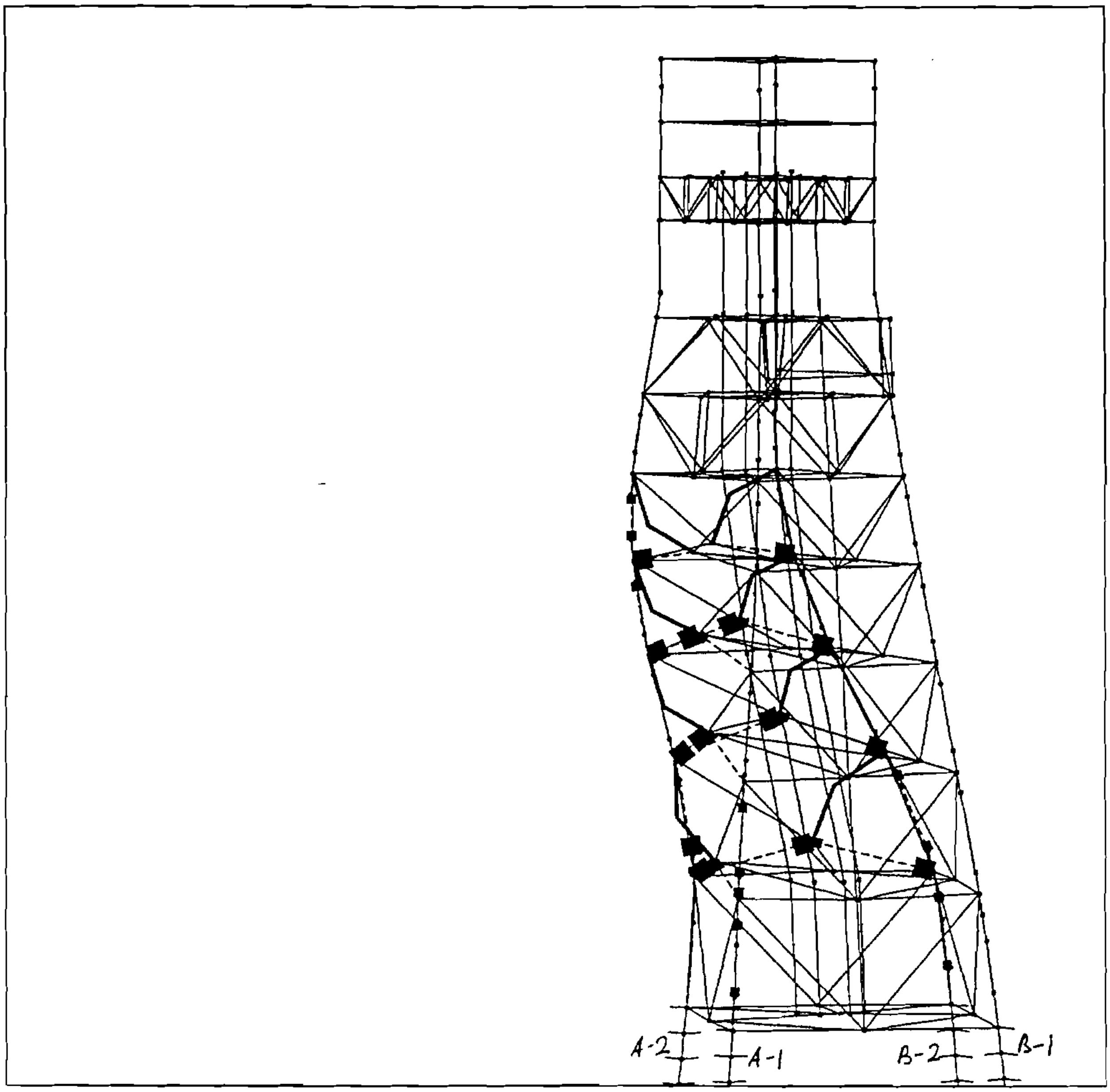


CAP z x At Lateral load = 3,880 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-16 Inelastic Events at Stage 5 (per Figure 6-11) — Diagonal Storm



At Lateral load = 4,020 kips

Inelastic Events Legend

————	Elastic	————	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	....	Fracture

Figure 6-17 Inelastic Events at Stage 6 (per Figure 6-11) — Diagonal Storm

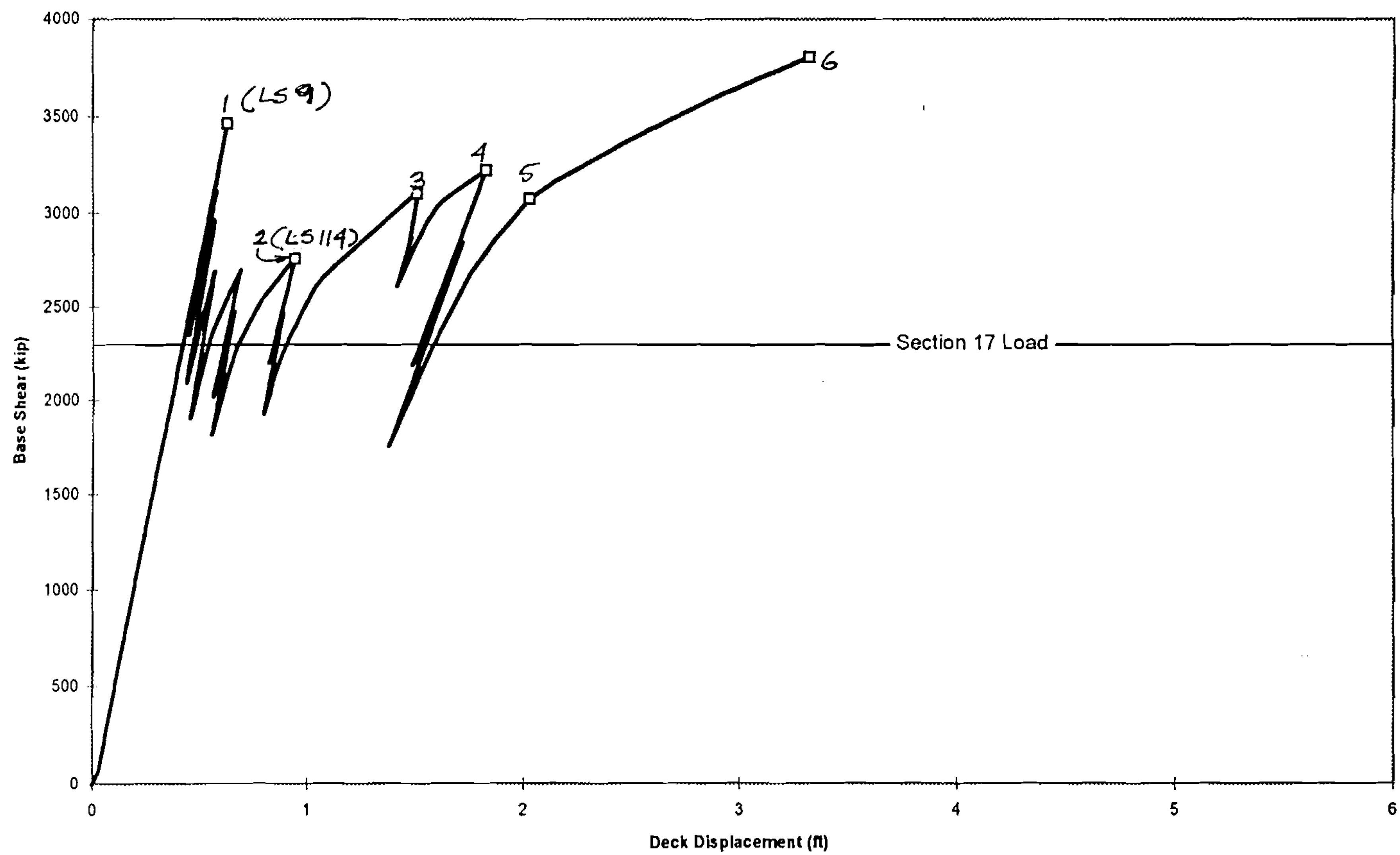
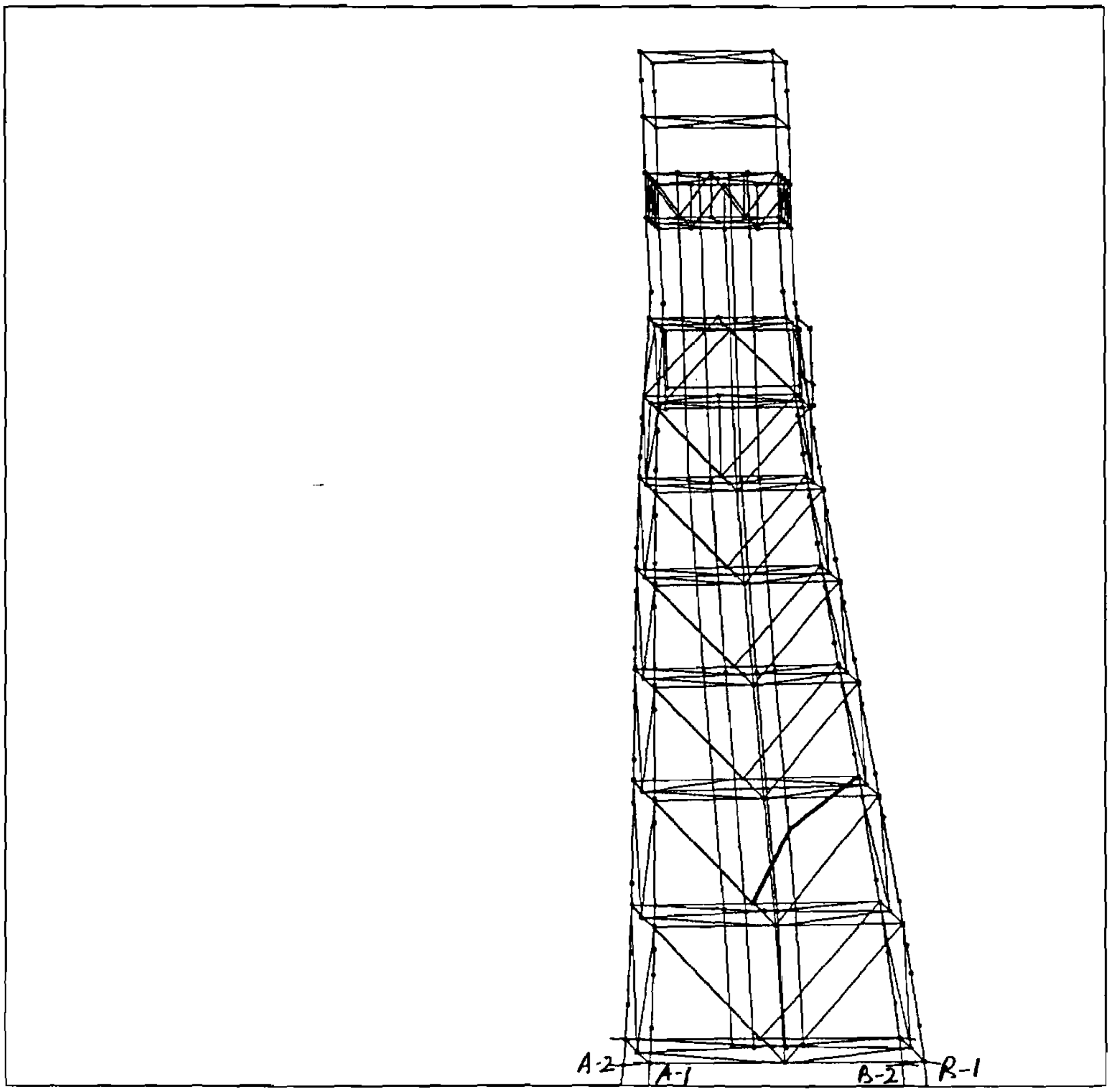


Figure 6-18 Fixed Base Case Load - Displacement Behavior — Broadside Storm



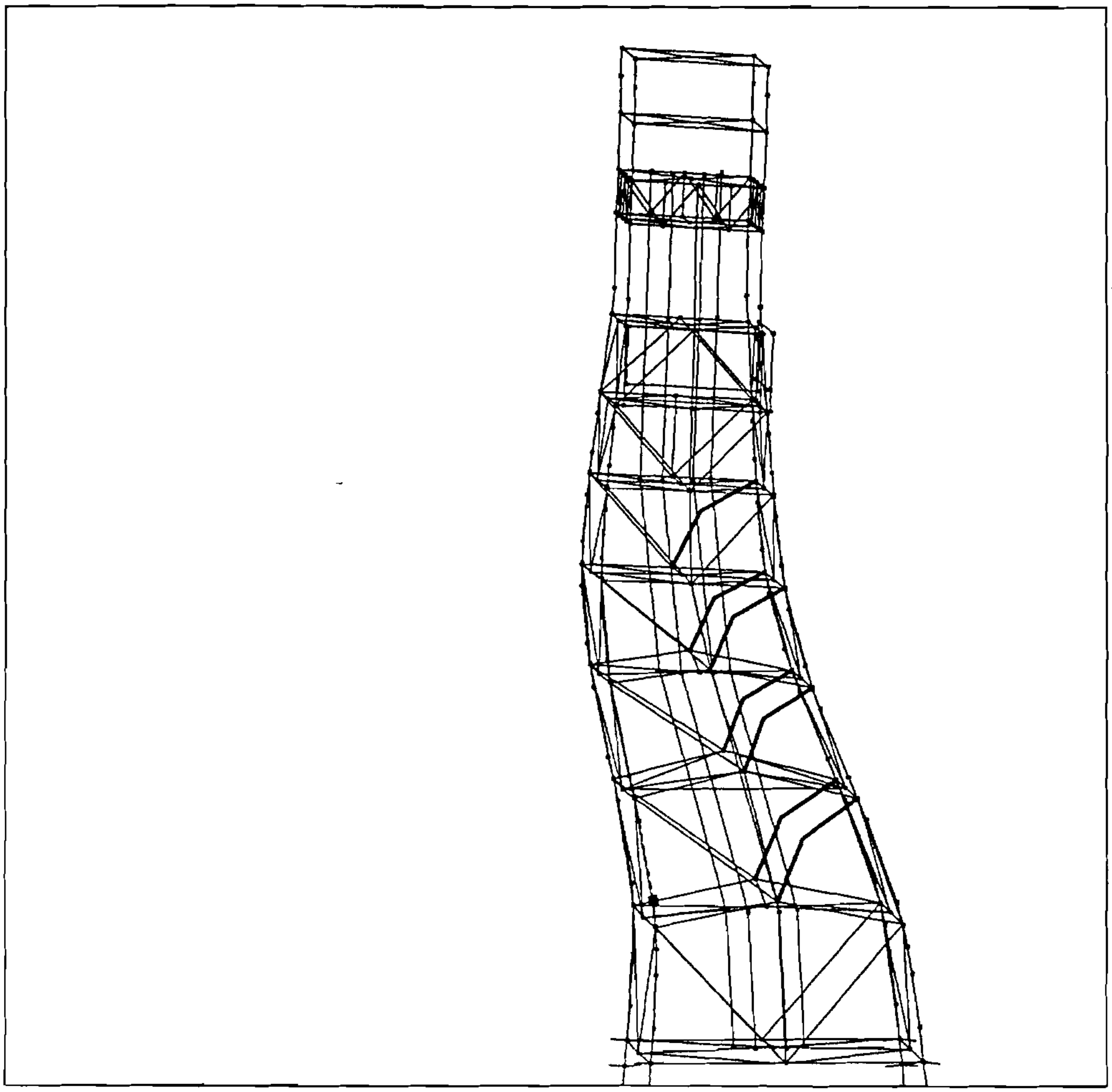
CAP

At Load Step 9, Lateral load = 3,465 kips

Inelastic Events Legend

————	Elastic	————	Strut Buckling
---------	Strut Residual	-.-.-.-.-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	....	Fracture

Figure 6-19 Inelastic Events at Stage 1 (per Figure 6-18) — Broadside Storm



At Load Step 114, Lateral load = 2,775 kips

Inelastic Events Legend

————	Elastic	━━━━	Strut Buckling
---------	Strut Residual	-·-·-·-·-	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	....	Fracture

Figure 6-20 Inelastic Events at Stage 2 (per Figure 6-18) — Broadside Storm

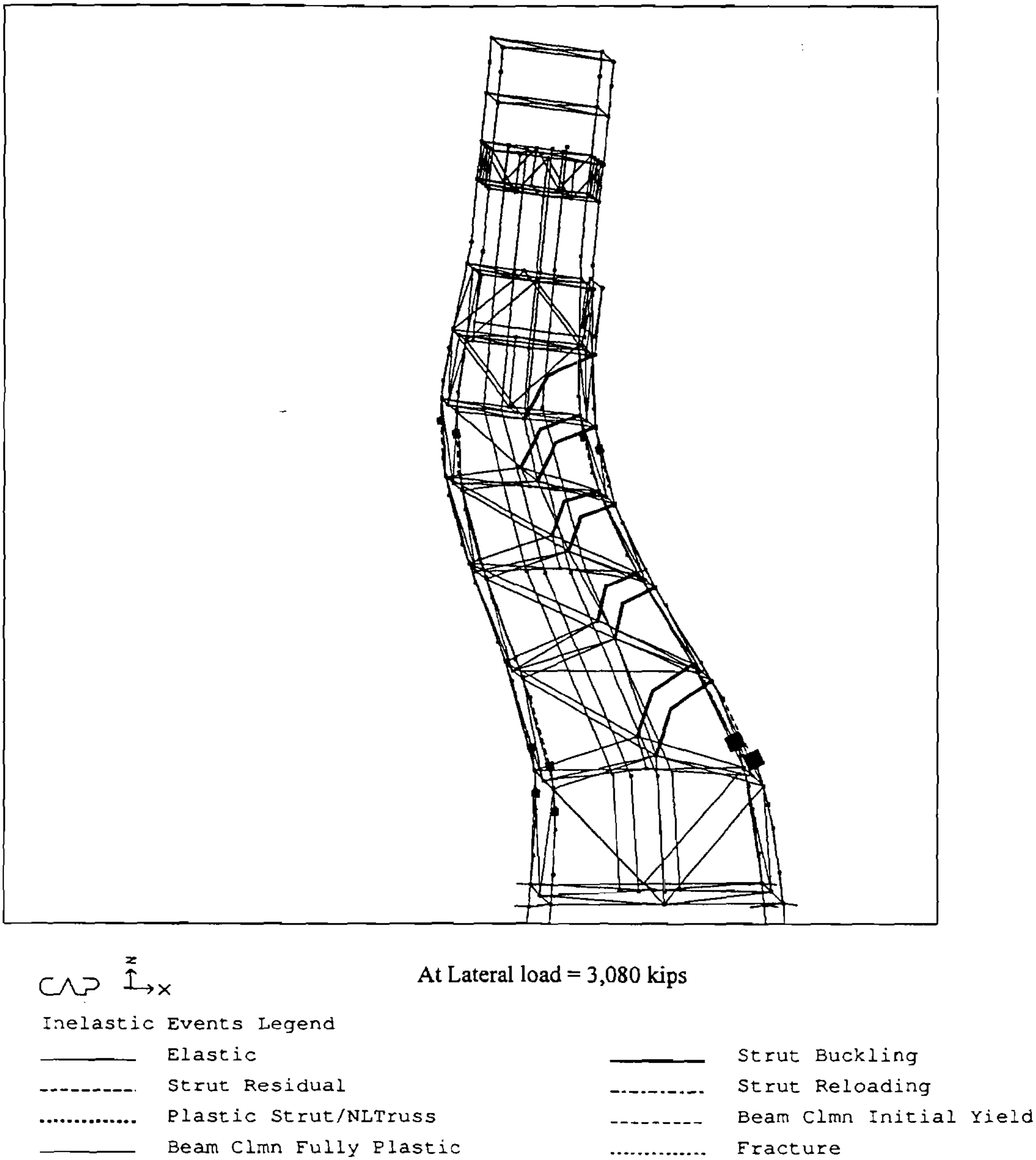
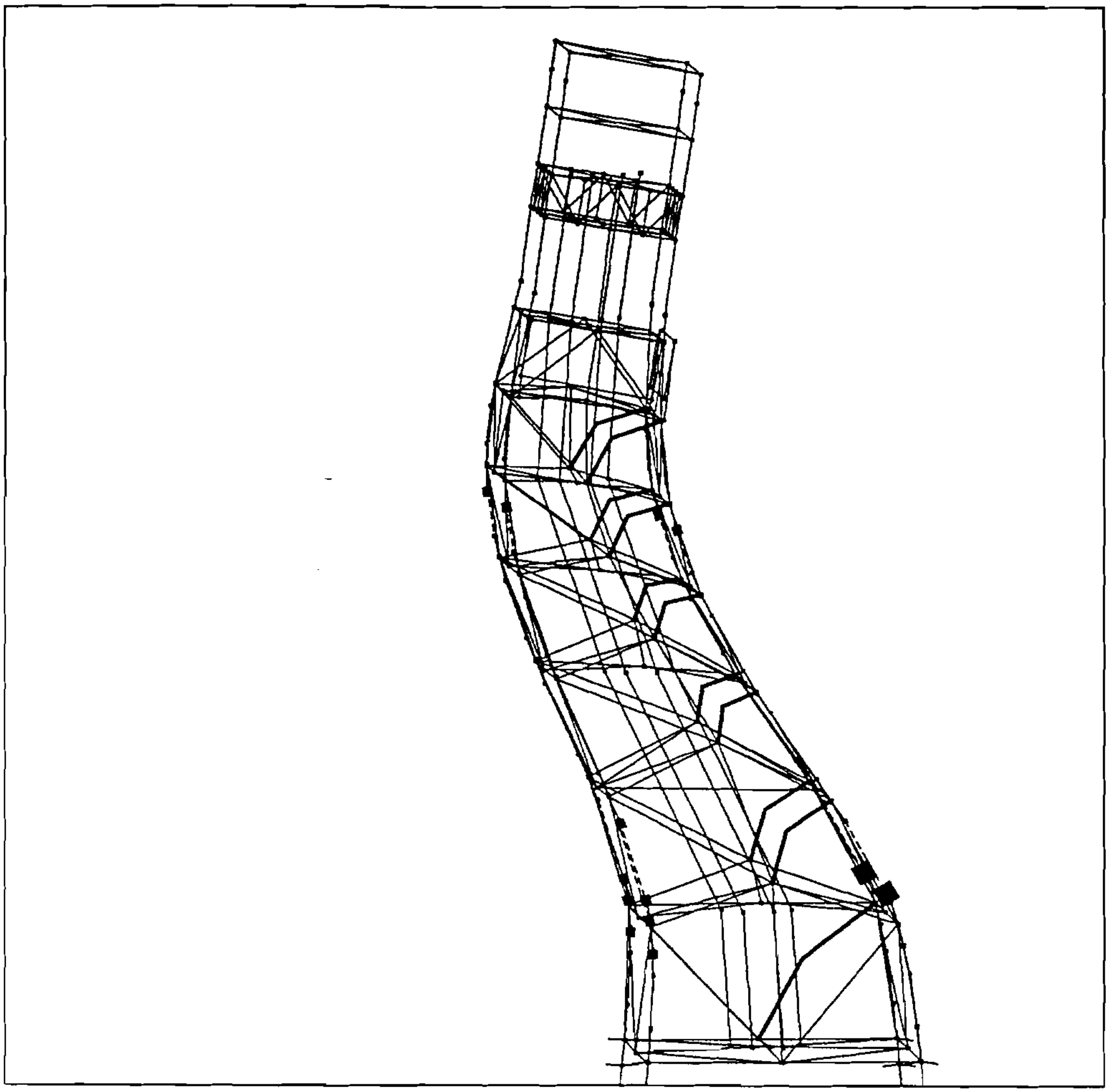


Figure 6-21 Inelastic Events at Stage 3 (per Figure 6-18) — Broadside Storm



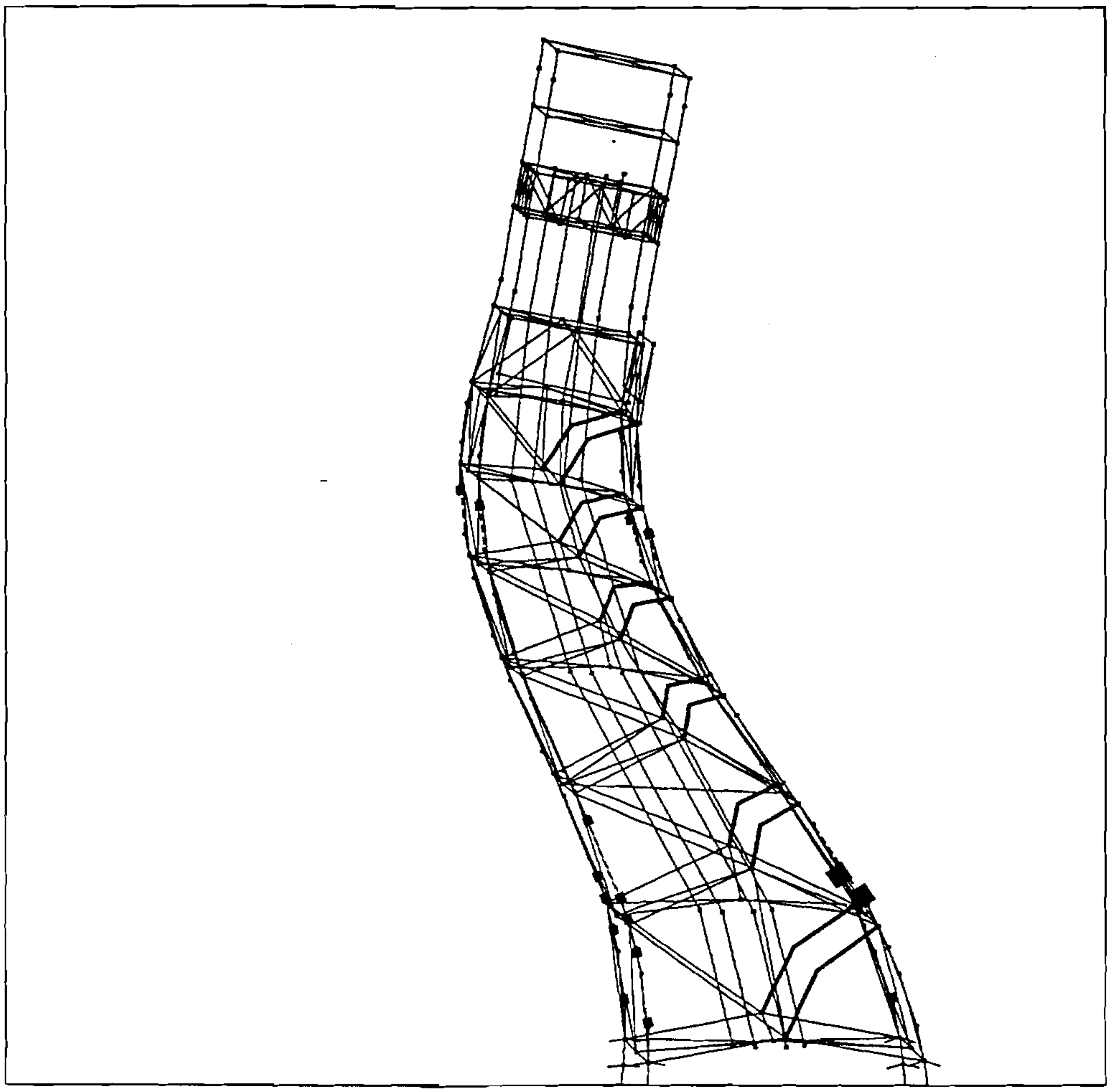
CAP z x

At Lateral load = 3,200 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-22 Inelastic Events at Stage 4 (per Figure 6-18) — Broadside Storm



At Lateral load = 3,060 kips

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Figure 6-23 Inelastic Events at Stage 5 (per Figure 6-18) — Broadside Storm

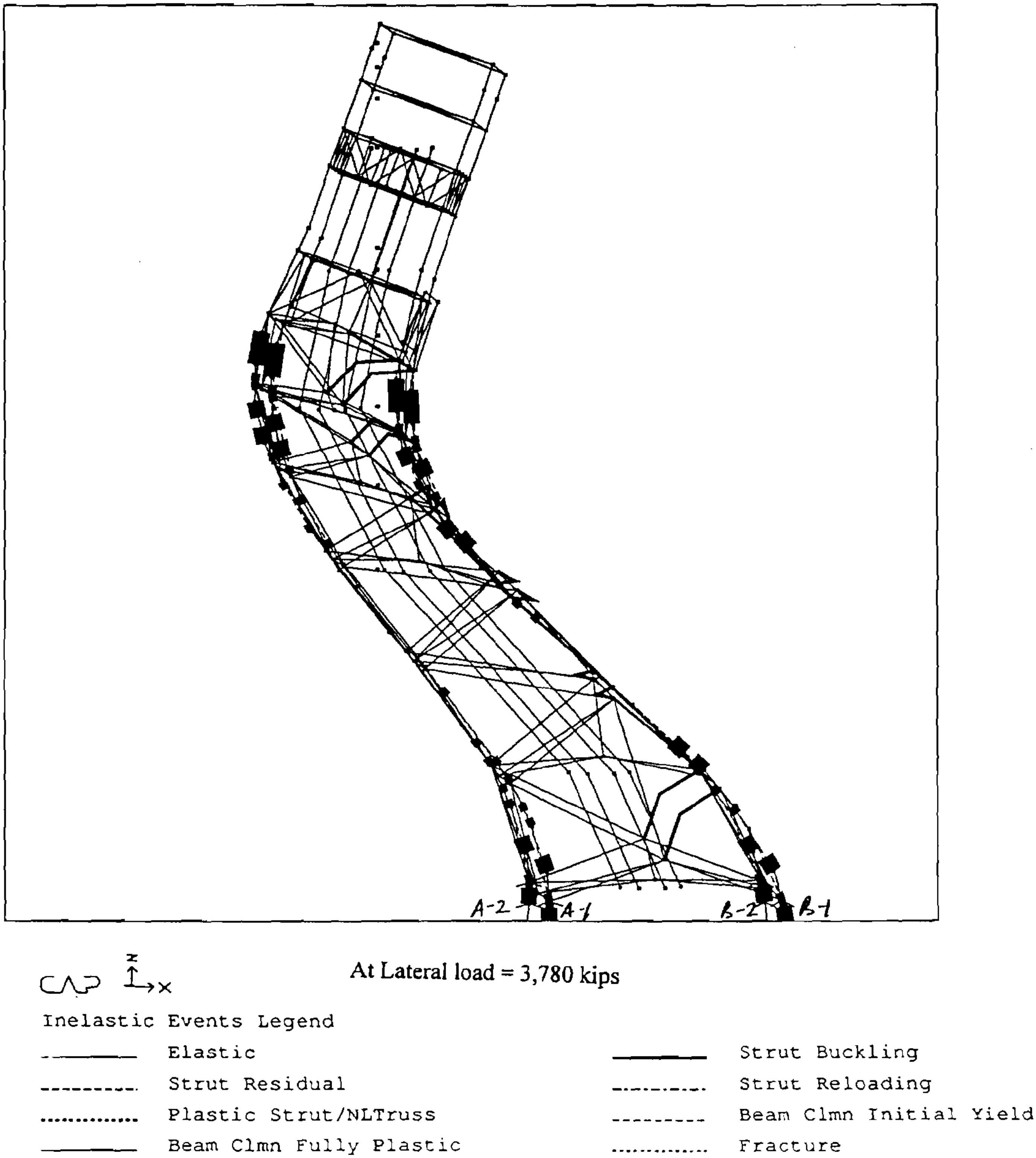


Figure 6-24 Inelastic Events at Stage 6 (per Figure 6-18) — Broadside Storm

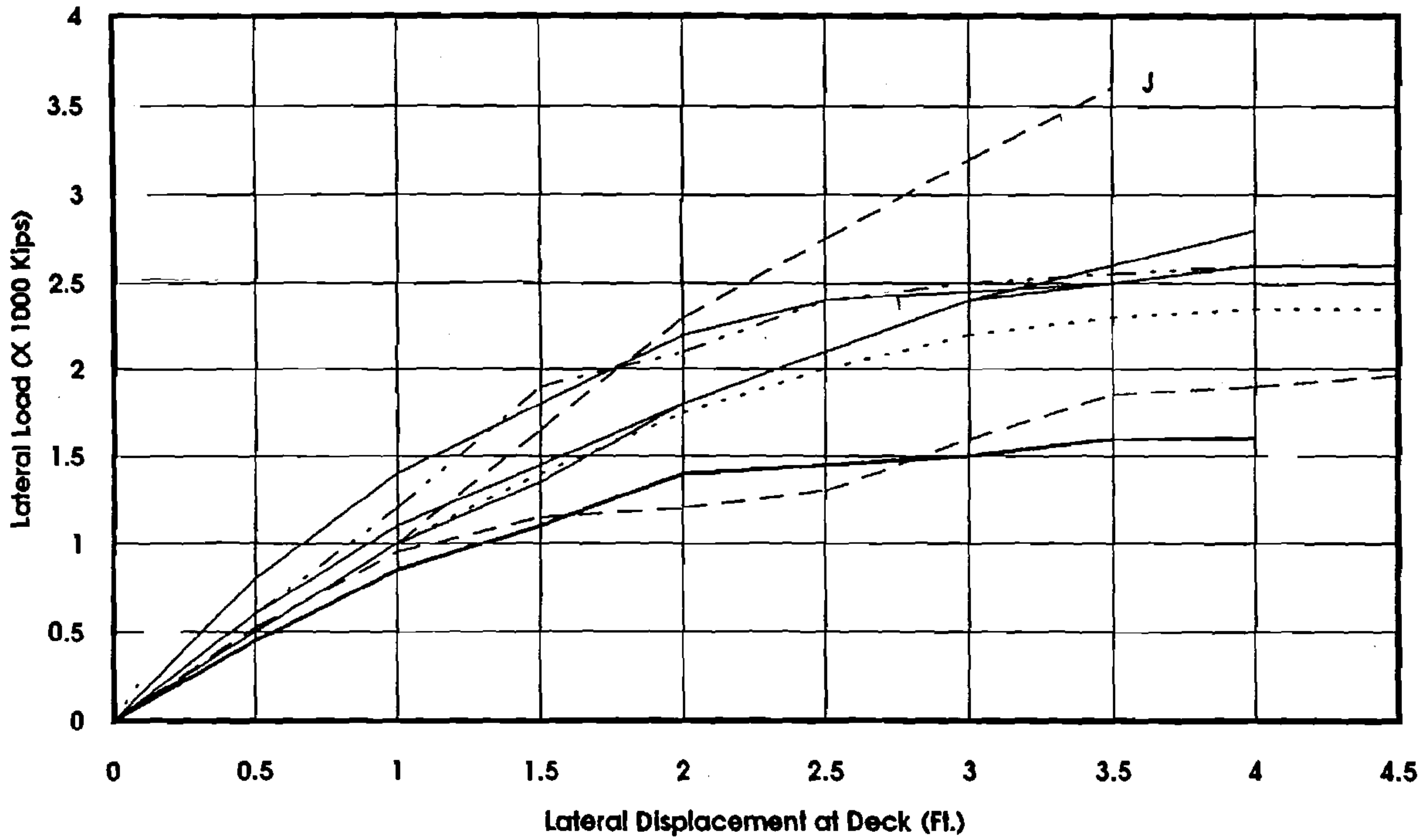


Figure 6-25 JIP Results — Base Case Load-Displacement Results (End On Storm)

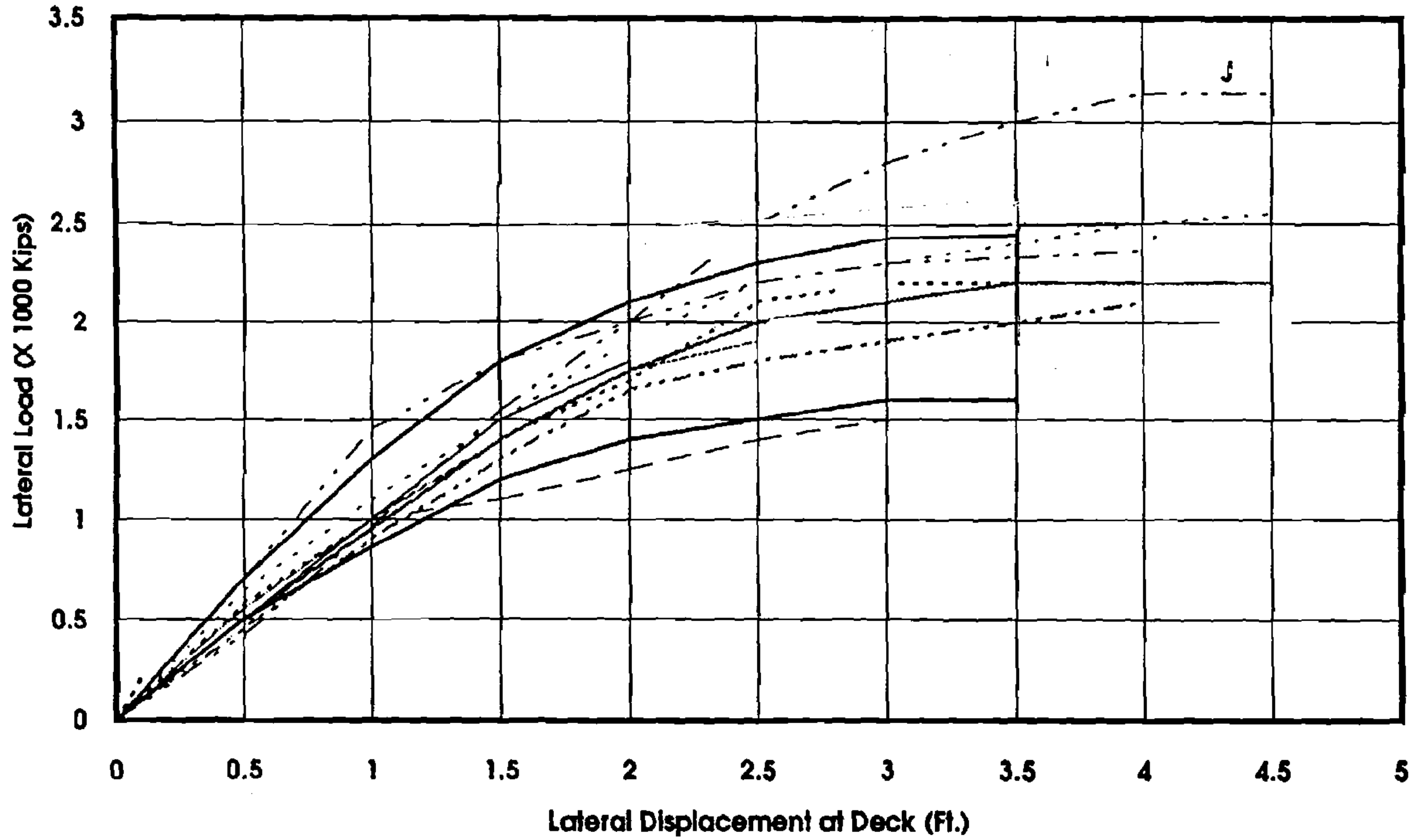


Figure 6-26 JIP Results — Base Case Load-Displacement Results (Diagonal Storm)

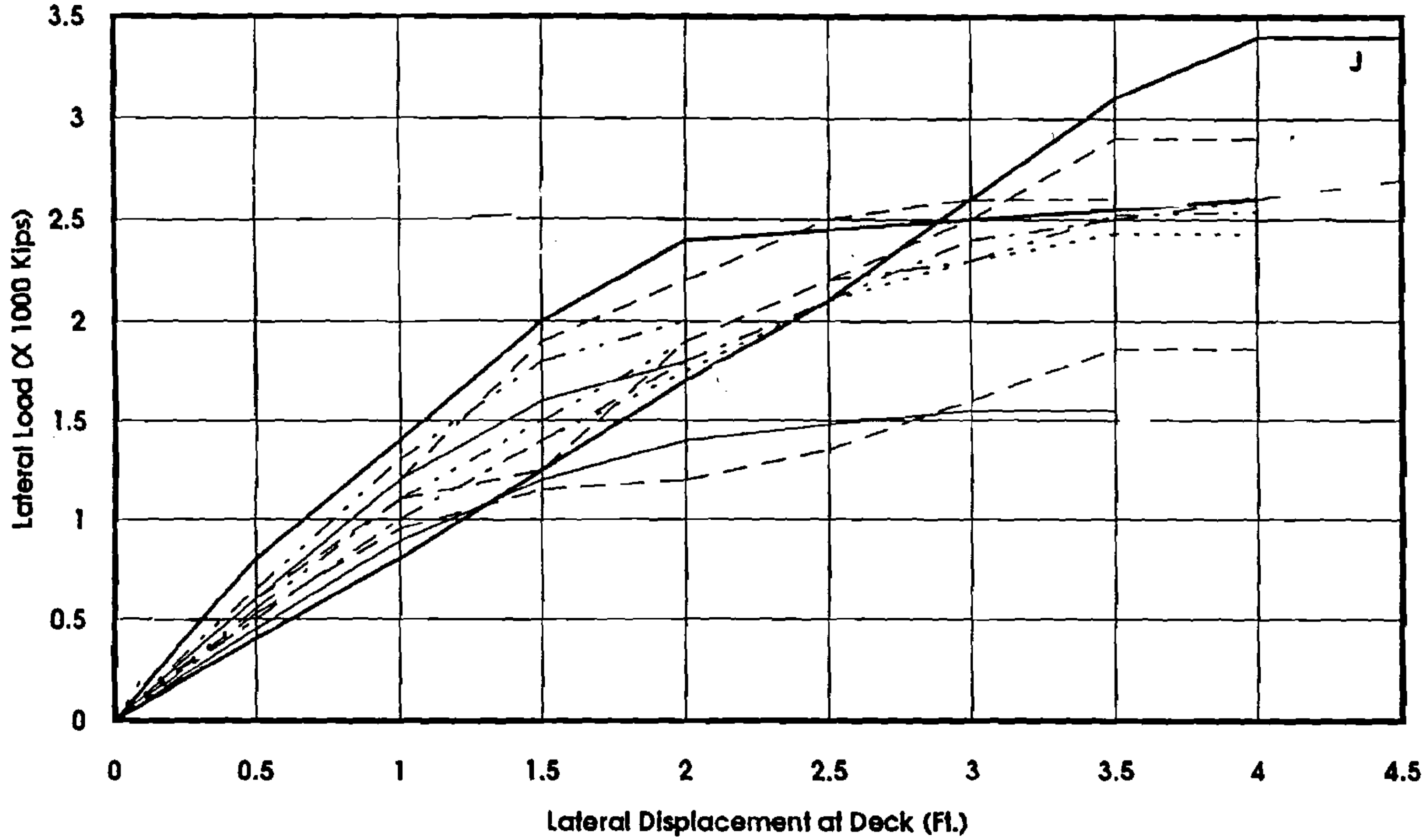


Figure 6-27 JIP Results — Base Case Load-Displacement Results (Broadside Storm)

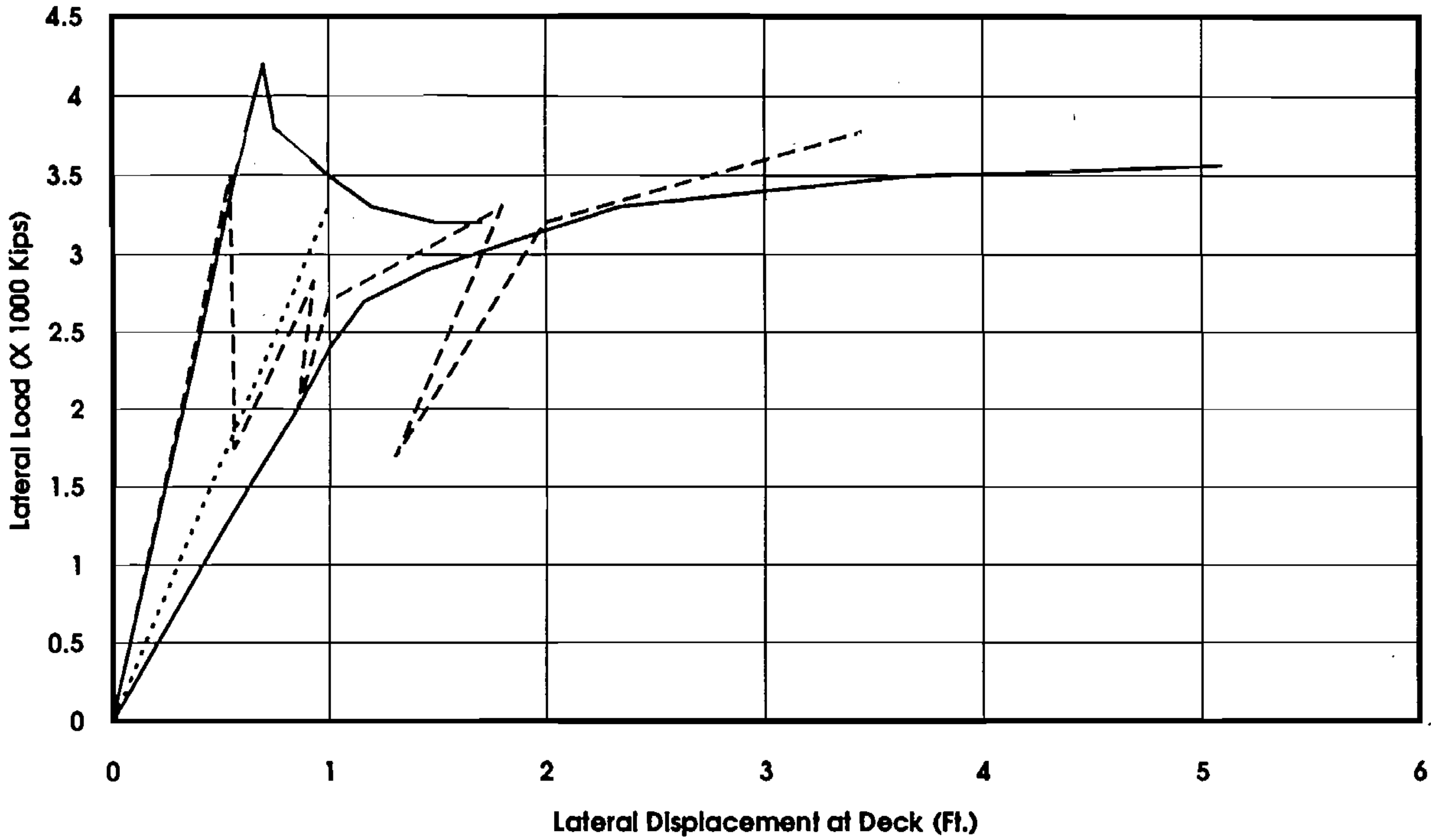


Figure 6-28 JIP Results — Fixed Base Case Load-Displacement Results (End On Storm)

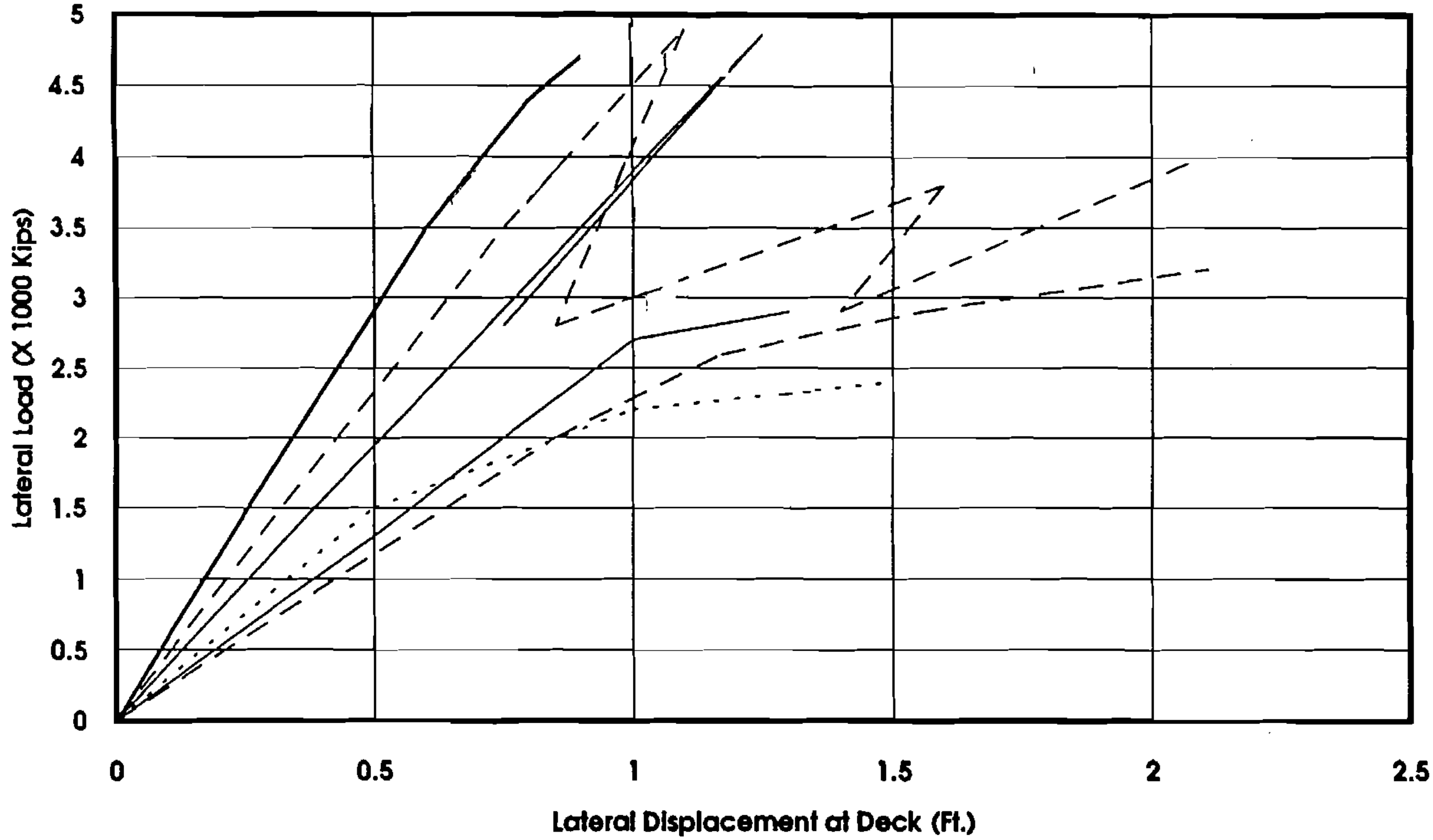


Figure 6-29 JIP Results — Fixed Base Case Load-Displacement Results (Diagonal Storm)

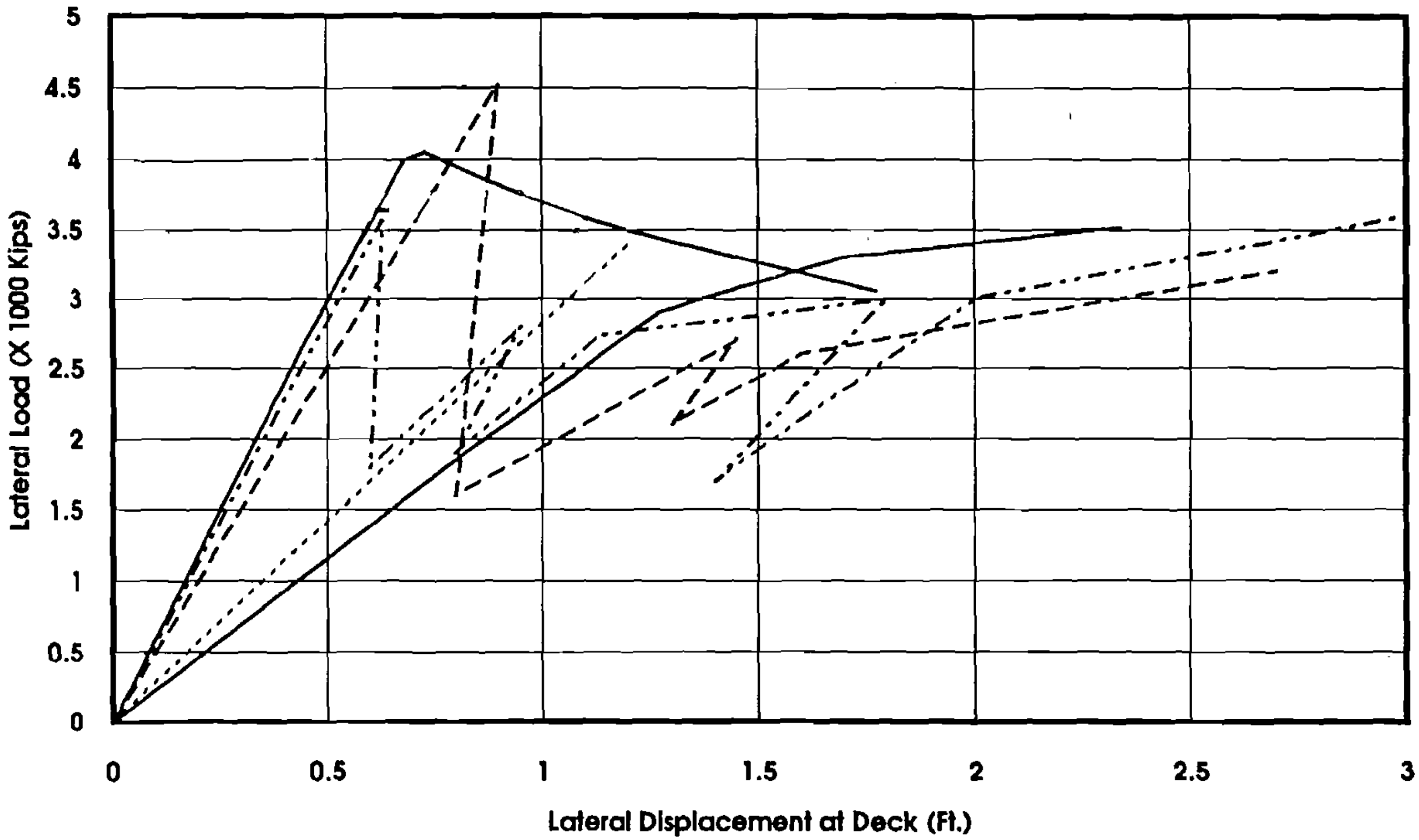


Figure 6-30 JIP Results — Fixed Base Case Load-Displacement Results (Broadside Storm)

This document has presented an illustrative example of the application of the API RP2A, Section 17 guidelines on one representative Gulf of Mexico platform. This document has been developed with two primary objectives. Firstly, the example is intended to provide engineers with a demonstration of each step of the assessment process that, when used in conjunction with the Section 17 commentary and supporting documentation (e.g., technical papers), should help improve ones understanding of the guidelines. Secondly, the results provided in this document can be used as a point of comparison against other analysis methods and software. In making such a comparison, the user should understand that these results do not represent the "correct answer" but are simply a good average of the results collected during the Joint Industry Project. The API Task Group has provided information regarding the range of results for this example problem that are considered consistent with the intent of the Section 17 guidelines [5].

It is important that the reader understand that the example provided should in no way be considered comprehensive. This example has addressed some of the issues that are generic to the platform assessment problem, however, applications of the Section 17 guidelines for other conditions (e.g., other structural configurations) may require different assessment procedures. It is the responsibility of the reader to become familiar with the Section 17 guidelines and the supporting documentation to understand these differences.

This example has addressed each step of the Section 17 assessment guidelines including ultimate strength analysis. The model used for the ultimate strength analysis provided in this example was very basic. As is the case with other API RP2A guidelines, it is prerogative of the user to define the scope of the analysis that is consistent with the requirements of the specific application in question. A variety of modeling options and analysis procedures are available (e.g., including dynamic effects, including biases in mean strengths of joints/braces/soil, use of variable pushover load patterns) that may be important in some cases. Also, in some instances, the nature of the response of the structure may dictate more sophisticated modeling and or analysis methods (e.g., modeling of damaged members). Conversely, more simplistic analysis methods may be sufficient in some cases and can result in ultimate strength estimates that are very similar to those generated by more rigorous analyses.

In cases where failure modes are developed simultaneously in different systems within the platform (e.g., brace and pile failure), it may be important to consider the variability in the definition of strengths for these components (i.e., COV's associated with jacket failure modes are less than those associated with pile capacity). In such cases, an assessment of the change in the estimate of ultimate strength resulting from the suppression of specific failure modes may be important.

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[1] American Petroleum Institute, 1996. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design, API RP2A, Twentieth Edition, Supplement No. 1, December 1996.

[2] Krieger, W., Banon, H., Lloyd, J., De, R., Digre, K.A., Nair, D., Irick, J., and Guynes, S., 1994. Process for Assessment of Existing Platforms to Determine their Fitness For Purpose, Proceedings, 26th Offshore Technology Conference, OTC No. 7482.

[3] American Petroleum Institute, 1995. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, API RP2A, Twentieth Edition.

[4] PMB Engineering Inc., 1994,a. Benchmark Analysis — Trial Application of API RP2A - WSD Draft Section 17, Final Report to the Joint Industry Project, December 1994.

[5] Digre, K.A., Puskar, F.J., Aggarwal, R.K., Irick, J., Krieger, W.F., Petrauskas, C., 1995. Modifications to and Applications of the Guidelines for Assessment of Existing Platforms Contained in Section 17.0 of API RP 2A. Proceedings, 27th Offshore Technology Conference, OTC No. 7779.

[6] PMB Engineering Inc., 1994-b. Trial Application of API RP2A - WSD Draft Section 17, Final Report to the Joint Industry Project, December 1994.

[7] Kallaby, J., Lee, G.C., Crawford, C., Light, L., Dolan, D., and Chen, J.H., 1994. Structural Assessment of Existing Platforms, Proceedings, 26th Offshore Technology Conference, OTC No. 7483.

[8] Chen, W. F., and Ross, D. A., 1977. Tests of Fabricated Tubular Columns, American Society of Civil Engineers, Journal of Structural Division, Vol. 103, No. ST3.

[9] American Petroleum Institute, 1989. Draft Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design, API RP 2A LRFD, First Edition, Washington D.C.

[10] Dier, A., and Lalani, M., 1995. Strength and Stiffness of Tubular Joints for Assessment/design Purposes, Proceedings, 27th Offshore Technology Conference, OTC No. 7799.

[11] SWRI, 1977. Model Testing of Braces to Develop Earthquake Analysis Criteria for Shell Oil Co. (Confidential).

[12] Grenda, K. G., Clawson, W. C., and Shinners, C. D., 1988. Large-Scale Ultimate Strength Testing of Tubular K-braced Frames, Proceedings, 20th Offshore Technology Conference, OTC No. 5832.

[13] Earl, C. P. and Teer, M. J.: "A Rational and Economical Approach to the Calculation of K-factors," Proc., 21st Offshore Technology Conference, OTC No. 6162, 1989.

[14] Hamilton, J. M. and Murff, J. D., 1995. Ultimate Lateral Capacity of Piles in Clay Proceedings, 27th Offshore Technology Conference, OTC No. 7667.

[15] PMB Engineering Inc., 1995. Further Evaluation of Offshore Structures Performance in Hurricane Andrew — Development of Bias Factors for Pile Foundation Capacity. Final Report to the API and Minerals Management Service (also API PRAC Report 94-81), May 1995.

[16] Aggarwal, R. K., Litton, R. W., Cornell, C. A., Tang, W. H., Chen, J. H., and Murff, J. D., 1996. Development of Pile Foundation Bias Factors Using Observed Behavior of Platforms During Hurrican Andrew, Proceedings, 28th Offshore Technology Conference, OTC No. 8078.

[17] Petrauskas, C., Finnigan, T.D., Heidman, J.C., Santala, M., Vogel, M.J., and Berek, G.P., 1994. Metocean Criteria/Loads Assessment of Existing Offshore Platforms, Proceedings, 26th Offshore Technology Conference, OTC No. 7484.

[18] PMB Engineering, Inc., 1996. Hurricane Andrew — Effects on Offshore Platforms, Phase II Final Report to the Joint Industry Project, January 1996. (confidential)

[19] Aggarwal, R. K., Dolan, D.K., and Cornell, C. A., 1996. Development of Bias in Analytical Predictions Based on Behavior of Platforms During Hurricanes, Proceedings, 26th Offshore Technology Conference, OTC No. 8077.

[20] PMB Engineering Inc., 1995. CAP — Capacity Analysis Computer Program, San Francisco, CA.