Trials Joint Industry Project

Benchmark Analysis — Trial Application of API RP 2A — WSD Draft Section 17

Volume I — Summary Report

by
PMB Engineering Inc.
San Francisco, CA

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Final Report

Trials Joint Industry Project

Trial Application of the API RP 2A-WSD Draft Section 17

BENCHMARK ANALYSIS

Prepared for

Minerals Management Service and Trials JIP Participants

Prepared by

PMB
ENGINEERING INC.

PMB Engineering Inc.

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Section 1
Introduction

1.1 BACKGROUND

API Task Group (TG) 92-5 developed a draft guideline called "API RP 2A-WSD 20th Edition, Draft Section 17.0, Assessment of Existing Platforms." The latest version of this document is dated April 29, 1994 with some particular revisions dated June 24, 1994. This document defines an assessment process as shown in Figure 1-1, which varies from that followed for a new design. The final type of analysis in the draft guideline is the "ultimate strength analysis" which determines the lateral load carrying capacity of a platform. Guidelines to establish the ultimate capacity are provided in the draft document. However, variability in the results of ultimate strength analysis may exist for a particular platform due to differences in interpretation of the draft guideline, different assumptions and computer modeling approaches used by engineers, and the different software available to the industry.

This draft guideline has not been yet officially endorsed by the API, and has been distributed to interested parties for comments by the TG.

The Minerals Management Service (MMS) and a number of interested participants (21 total) contracted PMB Engineering Inc. (PMB) to manage and coordinate a Joint Industry Project (JIP), called the TRIALS JIP, consisting of two parts as follows:

Part I: Trial application of the draft guideline in its entirety by the participants to their selected platforms.

Part II: Trial application of the ultimate strength analysis procedure of the draft guideline to a common platform by participants or any other interested organizations not participating in Part I, in order to determine the variability in the ultimate strength analysis results.

This report provides details of Part II of the project. Salient features of the common platform (hereafter called "benchmark platform") and results of ultimate strength analysis (hereafter called "benchmark analysis") by participants are summarized.

At the kickoff meeting held for the Part I participants of the Trial JIP project on January 19, 1994 at PMB/Bechtel, Houston offices, a Technical Advisory Committee (TAC) was formed to govern both Part I and Part II of the JIP. All companies participating in Part I of the project nominated one member to the TAC. Each TAC member was given one vote on all project matters.

A variety of candidate platforms were nominated for selection as a Benchmark Platform and discussed at the kickoff meeting. A variety of different configurations of typical offshore platforms was reviewed and discussed, with the final selection for the benchmark platform being a four-leg, four-well platform located in 157 ft water depth in the Gulf of Mexico. The platform was installed in 1970.

PMB developed the requirements of the Benchmark Analysis and produced a Benchmark Basis Document in agreement with the TAC. The Benchmark Basis Document provided the necessary background information for performing the analysis including details of the platform configuration and site conditions, as well as specific instructions on the types of analysis and results required of each participant. The Benchmark Basis Document was provided to the various companies interested in performing the Benchmark Analysis. This report summarizes the results provided by various companies in their Benchmark Documents.

PMB prepared Benchmark Analysis Draft Report (September 1994) and discussed the results with the TAC and Benchmark participants in the meeting on October 19, 1994. At the meeting the TAC voted for additional information from the participants to improve the database by elimination of missing information, gross errors or omissions, response to specific questions to identify reasons for variations, and agreed on the manner in which re-submittals will be incorporated by PMB. A copy of the PMB letter to the participants and response from some participants is provided in Appendix-C.

This report in its main sections summarizes the results provided by various companies in their original submittals and missing information in the Draft report. The effect of revised submittals by participants on the original set of results and response from the participants is given in Appendix-A.

The response of the API TG to participants' comments and queries, and the API TG interpretation of the applicable metocean criteria and wave force procedures is given in Appendix-B.

1.2 OBJECTIVES

The objectives of this portion of the Trials JIP were as follows:

- To assess variability in the ultimate capacity assessed by different companies
- To provide feedback to the API TG
- To provide training (learning the process) to the participating companies
- To establish relationships with contractors
- To trade notes with other organizations

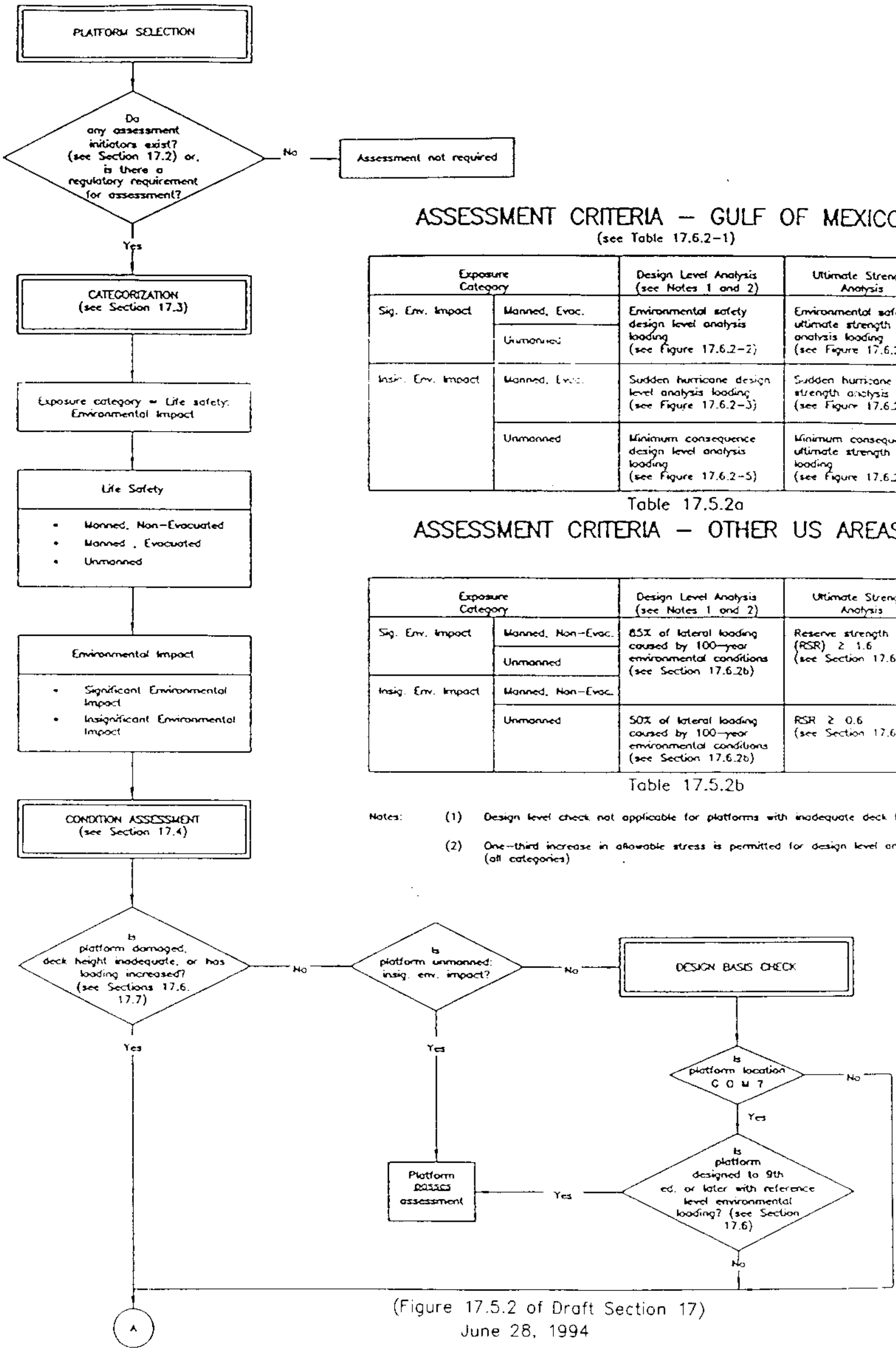
1.3 PROJECT PARTICIPANTS

At the kick-off meeting stage (January 19, 1994) 17 companies (6 operating companies and 11 engineering contractors) showed interest to perform Benchmark Analysis. Thereafter four more companies showed interest to participate.

Thirteen companies (5 operating companies and 8 engineering contractors) submitted their analysis to the project. Four companies provided re-submittal document by November 15.

These 13 companies (hereafter called "Benchmark Participants") are as follows:

AKER OMEGA
AMOCO
BARNETT & CASBARIAN/BOMEL, U.K.
CHEVRON
EXXON
HUDSON ENGINEERING
IDEAS
KVAERNER E & W/ DIGITAL STRUCTURES
MOBIL
OSI / ZENTECH
PMB ENGINEERING
SHELL
W. S. ATKINS, U. K.



ASSESSMENT CRITERIA – GULF OF MEXICO

(see Table 17.6.2-1)

Exposure Category		Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
Sig. Env. Impact	Manned, Evac.	Environmental safety design level analysis loading (see Figure 17.6.2-2)	Environmental safety ultimate strength analysis loading (see Figure 17.6.2-2)
	Unmanned		
Insig. Env. Impact	Manned, Evac.	Sudden hurricane design level analysis loading (see Figure 17.6.2-3)	Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2-3)
	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 17.5.2a

ASSESSMENT CRITERIA – OTHER US AREAS

Exposure Category		Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
Sig. Env. Impact	Manned, Non-Evac.	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		
Insig. Env. Impact	Manned, Non-Evac.		
	Unmanned	50% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b)	RSR ≥ 0.6 (see Section 17.6.2b)

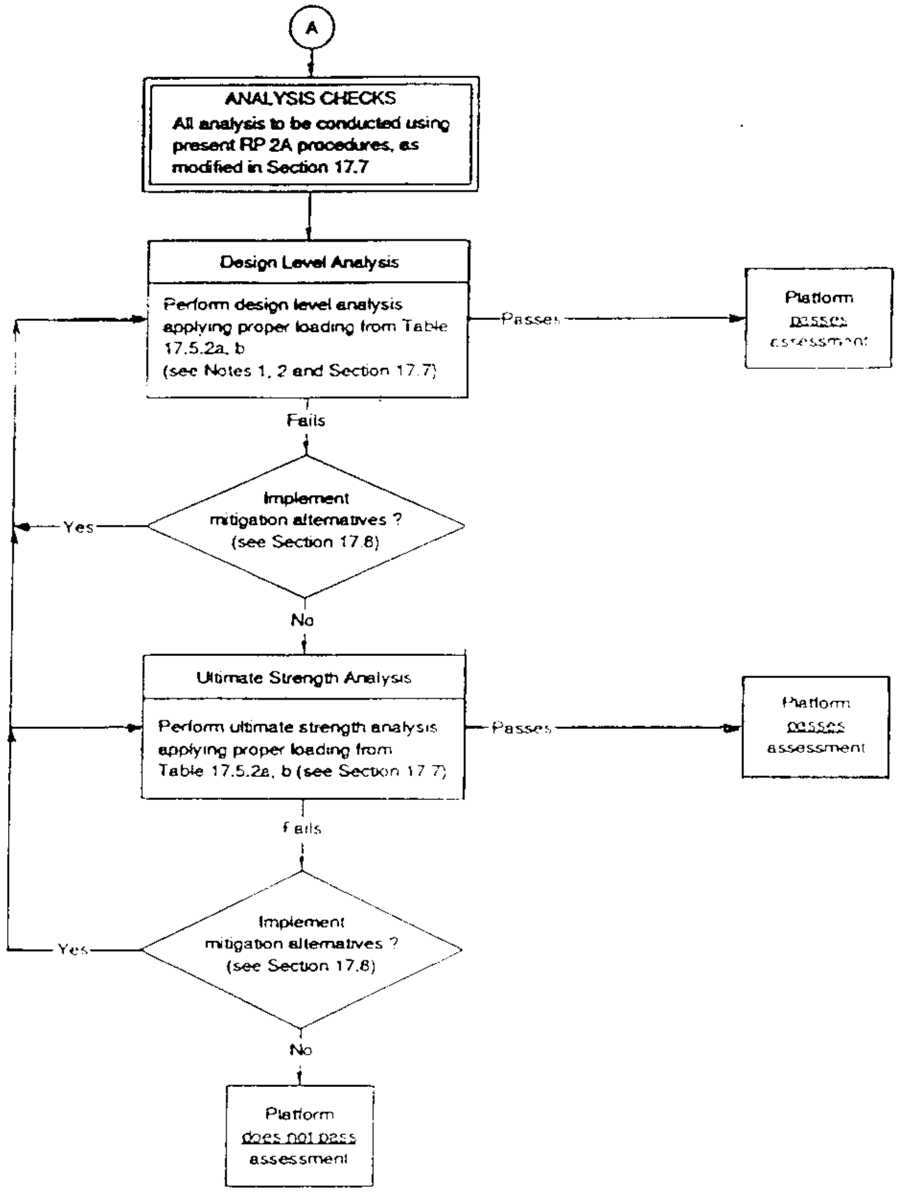
Table 17.5.2b

Notes: (1) Design level check not applicable for platforms with inadequate deck height.

(2) One-third increase in allowable stress is permitted for design level analysis (all categories)

(Figure 17.5.2 of Draft Section 17)
June 28, 1994

Figure 1-1 Section 17 - Platform Assessment Process Metocean Loading



(Figure 17.5.2 (continued) of Draft Section 17)
April 29, 1994

Figure 1-1 Section 17 - Platform Assessment Process Metocean Loading (continued)

Section 2
Information to Participants

2.1 BENCHMARK BASIS DOCUMENT

The participants were provided with platform orientation information, deck live load information, a complete set of required structural drawings (11" x 17"), pertinent parts of the soil report, and deck equipment views with the Benchmark Basis Document dated February 24, 1994. The document included details of project organization, analysis and documentation requirements for participation in the project. Two tasks were identified for the participants as follows:

Task A: Ultimate strength analysis of the benchmark platform by application of the API Section 17 Draft Guidelines. This task was required.

Task B: A critical review of the draft guideline, as applicable to the ultimate strength analysis, with emphasis on completeness, clarity, complexity, and suggestions where possible. Any typos or other errors should be identified. This task was voluntary.

The Benchmark Basis Document mentioned the following:

- Environmental conditions used in the benchmark analysis should be based on the sea state information contained in the Section 17 draft guideline and not from any other source (e.g., site-specific metocean study).

- API RP 2A-WSD, 20th Edition shall be used as the assumed "current edition of API RP 2A" referenced in the draft guideline.

- The number of wave approach directions for ultimate strength analysis will be defined by the participants themselves using the information contained in the draft guideline.

- The analysis shall be performed on a 3-D computer model of the benchmark platform. In general, the description given in the draft guideline for modeling (linear or nonlinear element types, soil modeling, etc) and approach (pushover, member removal, etc.) for the ultimate strength analysis shall be used. Participants were given option to deviate from the draft guideline to meet requirements of their software or for improved modeling. In such cases, the participants were to identify the different approaches followed.

- The nonlinear member types (elastic-plastic, strut, etc.) used in the model and formulas used (actual formulas or references to the equations in the API RP 2A or other publications) for member/ joint capacity equations were to be identified.

As a minimum, the following information was required from the participants for each wave approach direction analyzed:

- Reference level load (load corresponding to the 100-year seastate criteria) acting on the platform
- Ultimate strength of the platform
- Reserve Strength Ratio (RSR) of the platform

Additional optional information required by the project was as follows:

- Lateral load level when the first member experiences a nonlinear event
- Lateral load level at unity check of 1.0 (per RP 2A-WSD, 20th Edition)
- Ultimate strength analysis results for the fixed base case, assuming no piles below mud level and jacket fixed at the seabed
- Sequence and lateral load at failure of each component of the platform

Several participants had queries and requested additional information and identification of applicable parameters from RP 2A. This information was given to all the participants to provide more consistent computer models among participants. Revision 2 (dated April 12, 1994) and Revision 3 (dated April 20, 1994) to the Benchmark Basis Document included information on the following topics:

- Platform latitude and longitude
- Dead load of deck structure
- Projected area of deck
- Pile information
- Additional soil properties
- Risers information
- Marine growth
- Anodes

2.2 OVERVIEW OF BENCHMARK PLATFORM

The Benchmark Platform was installed in 1970 in 157 ft. water depth in the Ship Shoal area of the Gulf of Mexico. The platform has 4-production wells and a quarters facility. For purpose of the Benchmark Analysis, the platform is assumed to have a significant environmental impact if collapse should occur.

Figure 2-1 provides a key-plan of the platform. It is a four-legged platform with 30 ft. distance between legs at the work point elevation (El. + 16'). The platform has eight risers, four production wells, two boat landings and four barge bumpers.

The typical structural framing of the vertical frames of the platform is given in Figure 2-2. The framing in the two orthogonal directions is identical and consists of a K-brace system. The leg - pile annulus is ungrouted. Piles are connected to the jacket at Elev. (+) 13'.

The deck structure consists of four levels, with upper and lower decks. The lower deck extends from Elev. (+) 15'-6" to (+) 49'-6" and has two levels. The wellheads are located at the upper level of the lower deck. The total dead and live loads of the lower deck assembly is computed as 136 kips and 304 kips respectively. The upper deck structure extends from Elev. (+) 49'-6" to Elev. (+) 71'-3 7/8" and also consists of two levels. The upper deck carries all production and quarters facilities. The total dead and live loads for the upper deck assembly is estimated as 204 kips and 1,120 kips respectively.

The configurations of horizontal frames are given in Figure 2-3. At two levels, Elev. (-) 8' and Elev. (-) 97', no conductor framing is provided.

Pile details are given in Figure 2-4. Piles are 36 inches in diameter with a maximum thickness of 1.875 inch from the mud level to 80 ft. below. The piles penetrate 355 ft. below the mud level.

The variations of soil strength parameters with depth are given in Figure 2-5, which are taken from the McClelland Engineers, Inc. report of September 1969 for the Ship Shoal area. The soil consists of very soft-to-stiff gray clay from the mud level to 197 ft. below and stiff-to-very stiff gray silty clay from 225 ft. to 391 ft. The intermittent 28 ft. layer consists of very dense gray silty sand.

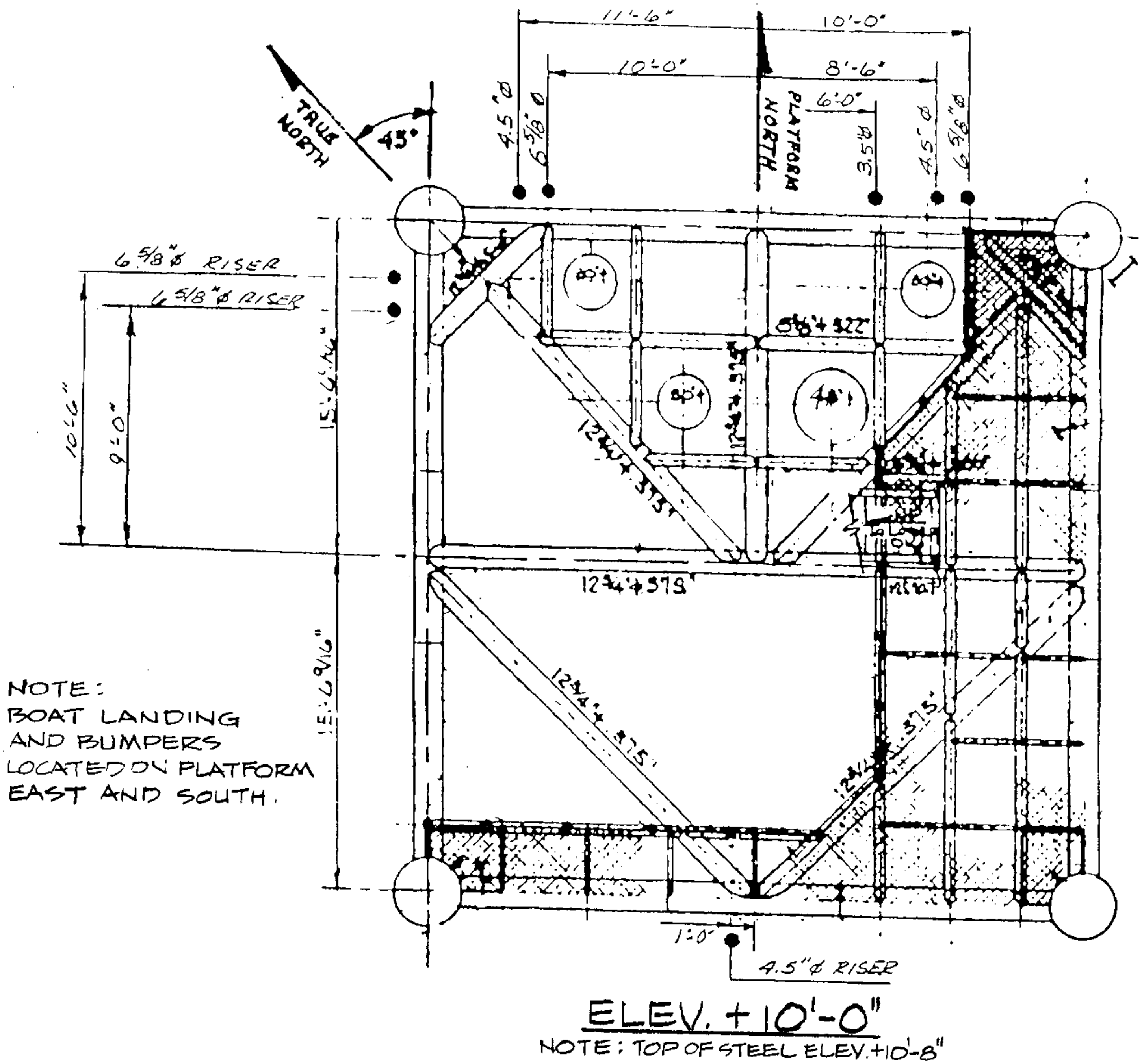


Figure 2-1 Key Plan - Benchmark Platform

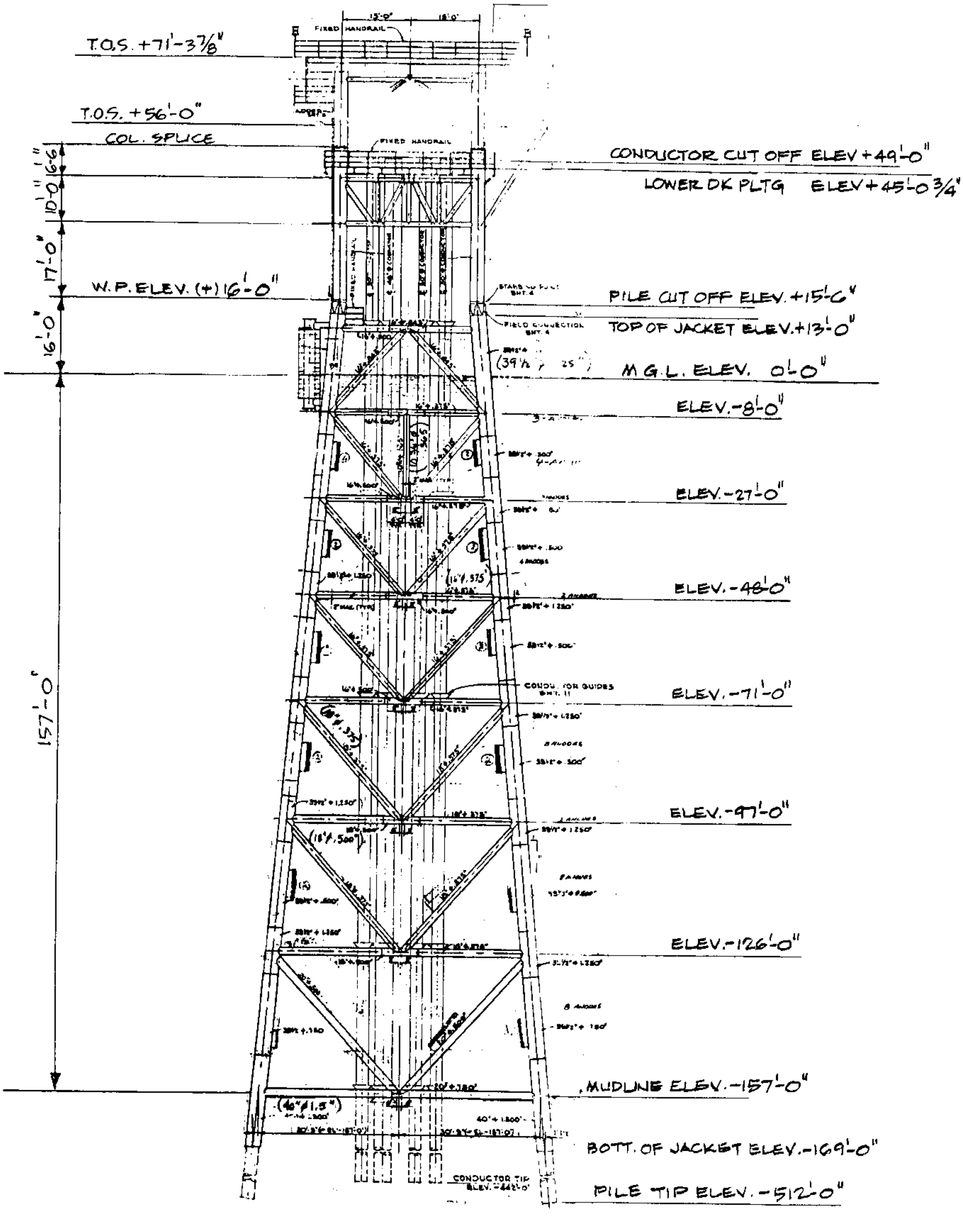
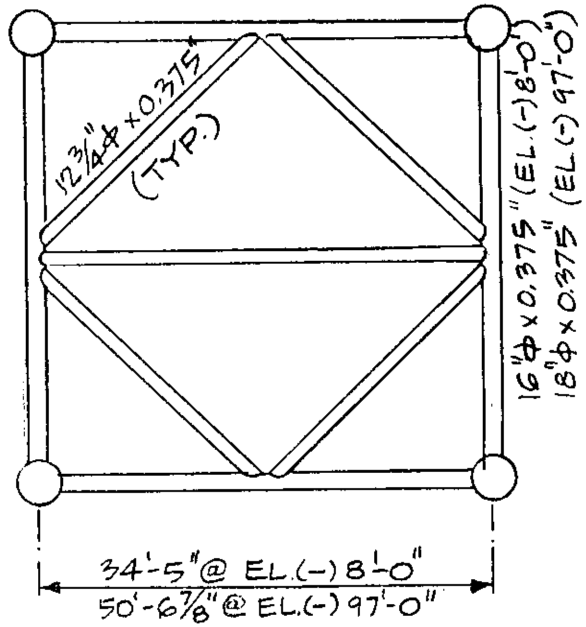
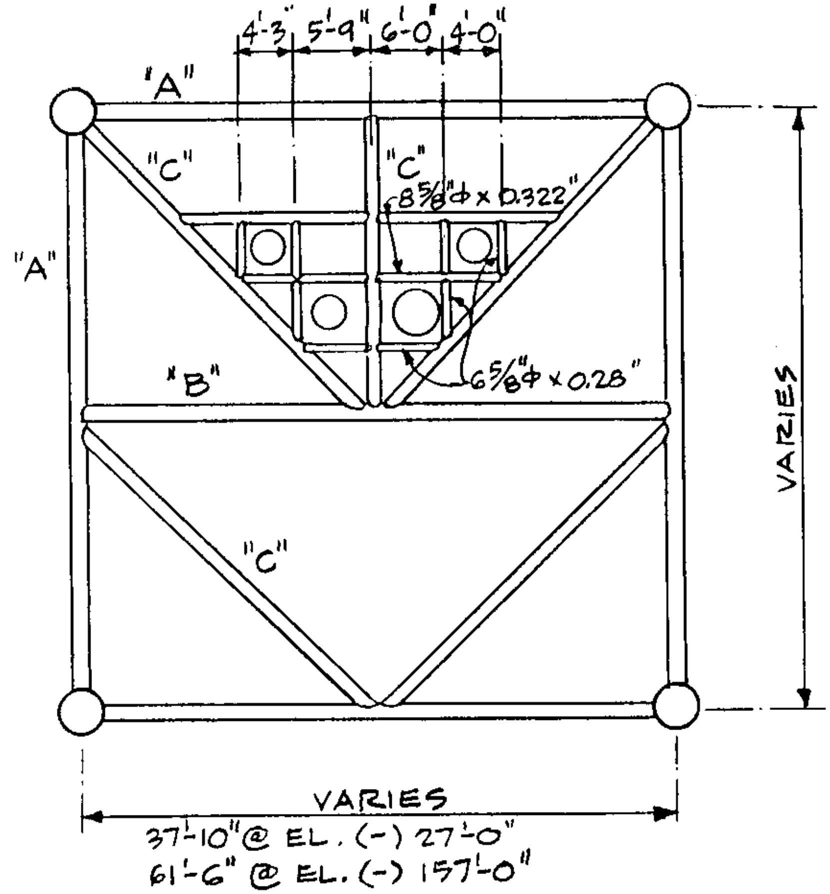


Figure 2-2 Typical Elevation - Benchmark Platform



EL.(-) 8'-0", EL.(-) 97'-0"



EL.(-) 27'-0", EL.(-) 48'-0", EL.(-) 71'-0"
EL.(-) 126'-0", EL.(-) 157'-0"

ELEVATION	A	B	C
(-) 27'-0"	16"Φ x 0.375"	14"Φ x 0.375"	12¾"Φ x 0.375"
(-) 48'-0"	16"Φ x 0.375"	14"Φ x 0.375"	12¾"Φ x 0.375"
(-) 71'-0"	16"Φ x 0.375"	16"Φ x 0.375"	12¾"Φ x 0.375"
(-) 126'-0"	18"Φ x 0.375"	16"Φ x 0.375"	14"Φ x 0.375"
(-) 157'-0"	20"Φ x 0.75	18"Φ x 0.375"	16"Φ x 0.375"

Figure 2-3 Typical Horizontal Framings - Benchmark Platform

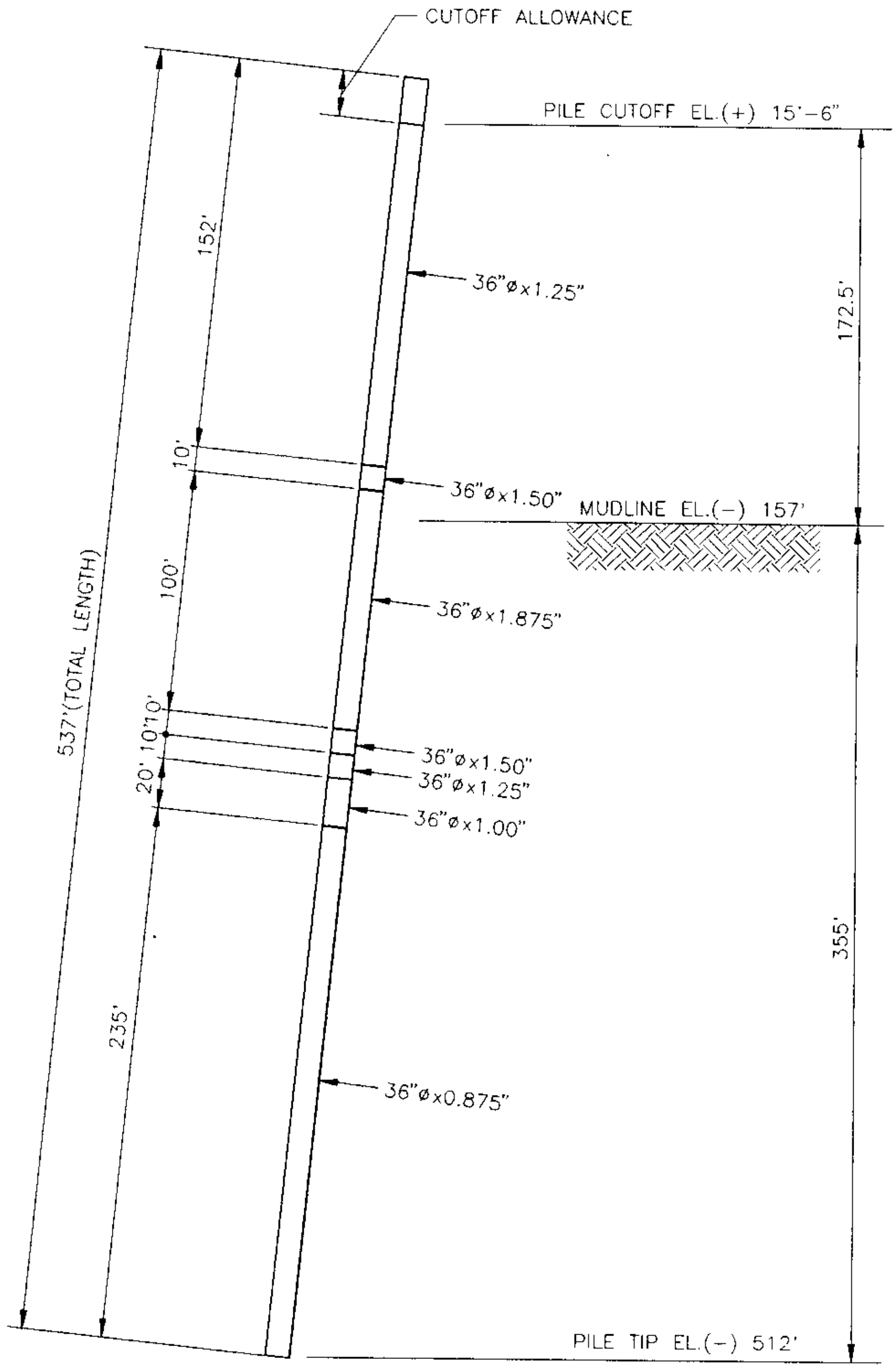


Figure 2-4 Pile Details - Benchmark Platform

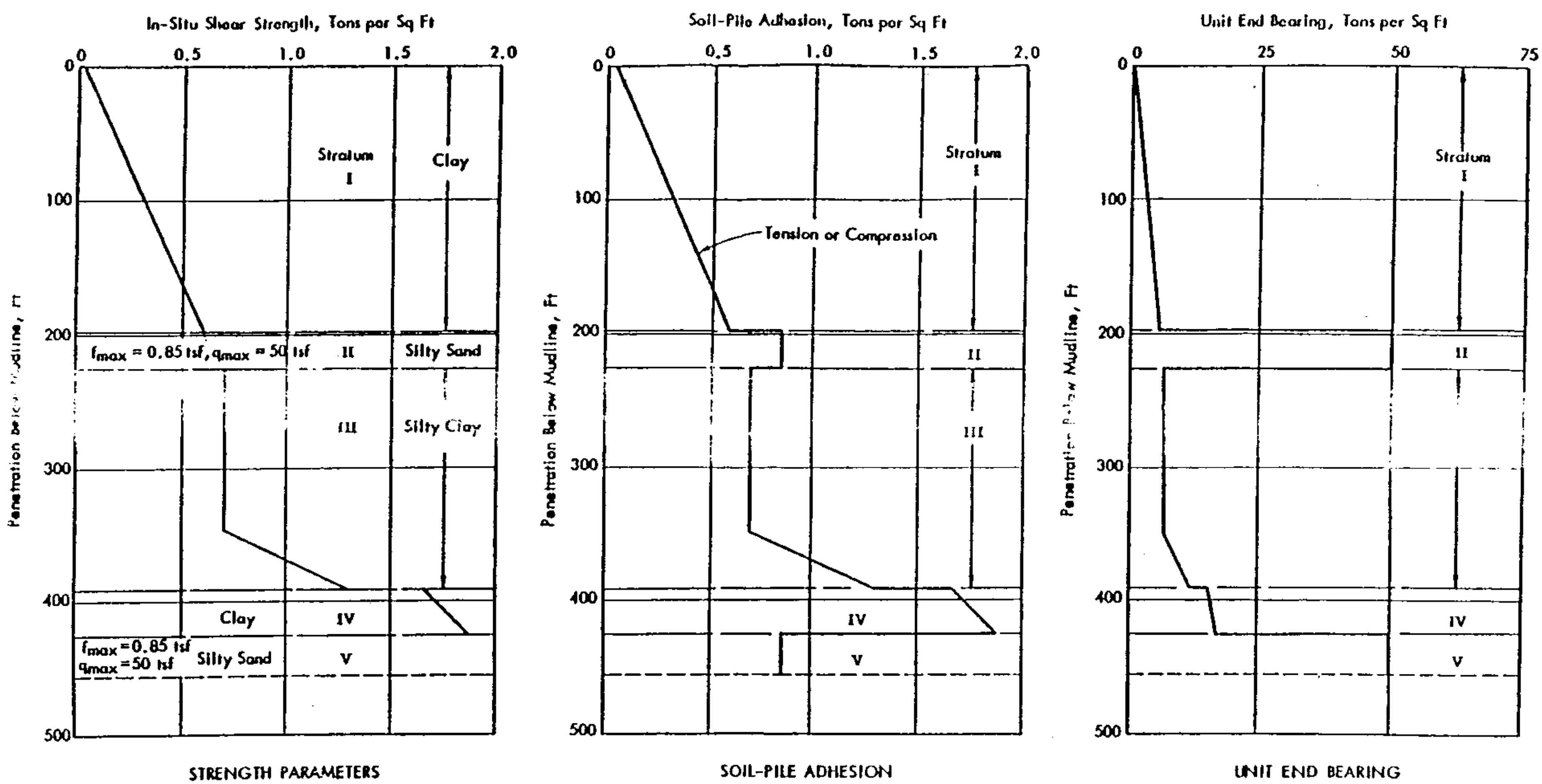


Figure 2-5 Soil Strength Parameters - Benchmark Platform

Section 3
Participants' Submittals

This section summarizes the information contained in the Benchmark Documents submitted by the participants. The information is summarized in the same format as suggested in the Benchmark Basis Document. The names of participating companies and the respective non-linear ultimate strength analysis software used have not been identified in this Summary Report. The participants are called Participants A to M in this report.

Following the Final Meeting several participants provided missing information in the Draft Report, and four participants (A, B, D, K) provided resubmittal documents. The Tables 3-1 to 3-9 in this section have been updated for the missing information and other clarifications from the participants but not for changes in the results due to elimination of "errors" in the original submittals. Figures 3-1 to 3-8 and 3-10 to 3-16 have not been updated. Figures 3-9 and 3-17 to 3-20 have been revised to reflect the changes

Abbreviated copies of the participants' submittals and re-submittals are provided in Appendices D and E in the Volume II.

3.1 ENVIRONMENTAL CRITERIA

The platform is located in the Gulf of Mexico and identified to have significant environmental impact upon its failure for purpose of the Benchmark Analysis. Therefore per Figure 1-1 and Table 17.6.2-1 (Draft Section 17), the FULL POPULATION HURRICANES metocean criteria is applicable for ultimate strength analysis. Thus the wave height and storm tide given in Figure 17.6.2.2-a are applicable. From Table 17.6.2-1, the associated wave period of 13.5 sec., current speed of 2.3 knots, and wind speed (1 hr @ 10 m) of 85 knots are noted. The applicable wave and current directions are per Figure 2.3.4-4 and Figure 2.3.4-5 of RP 2A, 20th Edition, respectively.

The majority of participants selected 3 directions for performing ultimate strength analysis. Participants documented approach angles in various ways: from True North, from Platform North, or from computer model X-axis. In this report and Figure 3-1, the directions are reported with respect to the TRUE NORTH (or Grid North) and the three directions, which were used by most participants, are labeled as Direction 1 (225 degrees from True North), Direction 2 (270 degrees from True North), and Direction 3 (315 degrees from True North). The parameters and results presented by participants have been reviewed to match these directions. Where the approach direction did not match these, the actual direction is mentioned in Tables 3-1 to 3-3. The results provided for any other direction have not been presented in this summary report.

Several participants found that the wave impacts the lower deck structure. The projected deck areas were provided to the participants. In order to ensure consistent use of Cd values given in the Table C.17.6.2-1 (for computation of wave/current platform deck forces), the values for "Moderately Equipped" deck type for the Lower Deck (first and second deck) and for a "Heavily Equipped" deck type for the Upper Deck (third and fourth decks), were provided in the Benchmark Basis Document.

The metocean parameters (wave height, current speed) and total base shear on the platform for the Section 17 (ultimate strength) criteria and for the RP 2A, 20th Edition criteria are given in Tables 3-1 to 3-3 for each of the selected directions. Where information was available, the wave-in-deck load values are also provided in the tables. The mean, standard deviation and coefficient of variation (COV) values are also provided for each parameter and load level. The information presented in these tables is discussed below:

- Wave Approach Directions: Tables 3-1 to 3-3 indicate that results provided by 11 participants matched the directions in Figure 3-1. Participant M selected approach directions for analysis that are the same as in Figure 2.3.4-4 (RP 2A, 20th Edition), i.e., 20 degrees clockwise to the others. Participant A's approach directions differed from the others.

- Wave Height (Section 17): A majority of participants selected the 68 ft. wave height from Figure 17.6.2-2a (Section 17). Figure 3-2 presents the variation of wave height for the three directions. The values selected for Direction 1 (COV = 7%) varied more widely among the participants than those for the other two directions. Participant M did not provide these values. The wave height values of participant A differ significantly from those of the other participants. Participant F used the same wave height (68 ft.) in two directions (Directions 2 and 3). (See Appendix C for discussions by participant H.)

- Current Speed (Section 17): The in-line current speed values with the wave direction, where information was available, are noted in the tables. A significant variation is noted for Direction 1 (COV = 28%).

- Wave Height (20th Edition): A majority of participants picked a 63 ft. wave height from Figure 2.3.4-4 (RP 2A, 20th Edition). Figure 3-3 presents the variation of wave height for the three directions. The values selected for Direction 1 (COV = 5%) differed more among the participants than for the other two directions. Participant J did not provide these values.

- Current Speed (20th Edition): The in-line current speed values with the wave direction are noted in the tables. A significant variation is noted for Direction 1 (COV = 28%).

- Wave-in-Deck Loads (Section 17): The values, where information was available, are given in the tables. A significant difference is noted in the wave-in-deck load estimates for all three directions (COV > 100%) among participants.

- Total Base Shear (Section 17): Figure 3-4 presents the variation of base shear for the 3 directions. There is a significant difference in participants' estimates (COV = 23%). In some cases, the resulting values differ significantly even when the wave height and current speed values are comparable. In general the estimates for participants A, D, and F were lower and those for participants E and G were higher than the results of other participants. The wave heights of participants I, J, and K were lower compared to those of participants B, C, H and L for the first two directions, whereas the load levels were comparable. The base shear for Direction 3 of Participant L was higher compared to that of Participants C, E, and H for the same wave heights.

  It is noted that a majority of participants selected Stream Function Theory of 7th Order. The benchmark documents indicate that 3 participants (E, F, I) used Stoke's 5th order wave theory and participant J used Airy's wave theory. In two cases, participants identified the limitation of their software for their selection.

- Wave-in-Deck Loads (20th Edition): The wave-in-deck load estimates of participants D and E are significantly lower and of participant K are significantly higher than those of the other six.

- Total Base Shear (20th Edition): Figure 3-5 presents the variation of lateral load for the 3 directions. A significant difference is noted in participant estimates. In some cases, the resulting values differ significantly even when the wave height and current speed values are comparable. The observation of low and high cases for this case are similar to those noted for base shear estimates per Section 17. Participant M reported highest values for all three directions.

The base shear results indicated no clear pattern of variation when the values are compared considering difference in magnitude of selected wave heights. A detailed interpretation of the causes for the observed differences was not in the scope of the project. However, besides the selection of metocean parameters, the largest differences in base shear magnitude are likely due to variability in use of the 20th Edition hydrodynamic force computation procedures (various coefficients and factors used, wave theory, current stretching), modeling differences, etc.

Following the project meeting held on October 19, 1994, the API TG WG3 developed their interpretation of the applicable metocean criteria and wave force procedures to the

Benchmark platform analysis meeting the requirements of the Benchmark Basis Document. The complete information from the API TG is given in Appendix-B.

3.2 3-D MODEL GENERATION

All participants performed analysis on three-dimensional models. However, the models generated differ significantly due to analysis procedures and software used. Some observations are presented below:

All participants except participant L modeled the primary jacket components as nonlinear elements. Participant L modeled them as linear elements and followed a member replacement (for members with unity check per API code check formula exceeding 1.0 with all safety factors removed) approach in their analysis.

Several participants found joint capacity to be critical and modeled K-braces based on joint capacity. Some participants found joint capacities were higher than the brace capacities. However, other participants found joints to be weaker than braces but did not consider their effect initially in their model.

Some participants modeled conductors as wave load elements, whereas others modeled them as linear beam or nonlinear beam column elements. Several participants modeled conductors below the mud level as lateral load carrying members.

A majority of participants modeled pile and soil springs with their model and performed an integrated analysis. Some participants modeled nonlinear pile-soil behavior by equivalent soil springs at the base of the jacket and followed an iterative process.

Some participants mentioned limitation of the software used in their modeling assumptions. Some participants mentioned including P-delta effect in their analysis.

3.3 SOFTWARE DESCRIPTION

Nine different nonlinear analysis software packages were used by the participants. In some cases, participants' software had integrated facilities for model generation, wave load computation, pile/soil analysis, and postprocessing of results. Whereas, in other cases, one or more of these features were not available and other software programs were used. The list of the nine software packages used for nonlinear analysis with the owner company names is given below:

ASADS	– IDEAS
CAP/SEASTAR	– PMB Engineering
EDP	– Digital Structures

KARMA	– ISEC
MicroSAS	– Hudson Engineering
RASOS	– W.S. Atkins, U.K.
SAFJAC	– BOMEL, U. K.
StruCAD*3D	– Zentech
USFOS	– SINTEF, Norway

A description of software programs used is included in participants' submittal provided in Appendices D and E.

3.4 ULTIMATE STRENGTH ANALYSIS RESULTS (REQUIRED)

Tables 3-4 to 3-6 summarize the required results from the ultimate strength analysis for the three directions. Nine participants performed analysis for all three directions, 2 participants (F and L) performed for 2 directions, and 2 participants (E and K) performed analysis for 1 direction.

These tables present the ultimate capacity, reserve strength ratio (RSR), and failure mechanisms. In addition, where the information was available, these tables present base shear values for Section 17 and 20th Edition criteria load levels, when the first member experiences an IR of 1.0 and when the first member has a nonlinear event. The mean, standard deviation and COV values for each quantity are given in the tables.

Load Level when First Member has an IR of 1.0 (Optional): Only 4 participants (B, D, H, and I) provided this information. Participant J provided very high values (obtained based on ultimate strength of the member) which are also the same as the load level when first member has a nonlinear event. Thus, these values are not included in tables and figures. Participant B computed the load level using the LRFD approach and its value differed significantly from those of participants D and H.

Load Level when First Member has a Nonlinear Event (Optional): This load level varied significantly among participants. In particular, Participant J values were found to be very high compared to all others.

Ultimate Capacity (Ru): Figure 3-6 presents a comparison of ultimate capacity values for the three directions. A significant spread in the values is noted. The ultimate capacity values varied between 1,500 kips and 3,600 kips for the three directions. Participants G and L determined their ultimate capacity to be the same irrespective of wave approach direction. In general, the values for the diagonal direction were 3 to 15 per cent lower than for the two orthogonal directions.

Reserve Strength Ratio (RSR): Figure 3-7 compares RSR values (generally provided by the participants) in the three directions. A significant difference is noted in the values among participants. The RSR values vary from 0.7 to 2.5 for Direction 1, from 0.6 to 2.2 for Direction 2, from 0.7 to 2.2 for Direction 3.

Some participants computed RSR using different load values. RSR is defined in Section 17.5.2 as "the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present RP2A procedures." Participant A used the Section 17 "design level" loading and participant E used the load level corresponding to the Section 17 ultimate strength metocean criteria. The tables indicate RSR values using their 20th Edition load levels.

Figures 3-8(a) to 3-8(c) provide 3-D presentation of variation of the 20th Edition reference load level and ultimate capacity values for the three directions. These figures indicate that there is no clear pattern of variation in three directions and the two quantities vary randomly among participants.

Component and Platform Failure Modes: Figure 3-9 presents a comparison of component and platform failure modes obtained by participants for Direction 2 (270 degrees from True North). The component failures obtained by the participants from the first member with a nonlinear event to formation of failure mechanism are identified in this table with shaded blocks. The platform failure modes identified by the participants are given in the bottom row of the table.

Participants established component failure modes and mechanism formation in the jacket structure (K-braces and jacket legs). Participant F found pile yielding and hinging as the only failure modes. Two participants (B and H) found yielding of the jacket leg and pile sections and established pile yielding to form their failure mechanism. Participant M found soil capacity to govern and did not find failure of any components of the platform. Seven participants (A, C, D, E, G, I, and L) found inadequate soil capacity to define failure in addition to other nonlinear events in the jacket or pile.

Load-Displacement behavior: Figures 3-10 to 3-12 present the load-displacement behavior of the platform by different participants. The patterns of variations for the initial stiffness (linear part), stiffness change with component failures and the ultimate capacity values are significantly different among participants. In general, the ultimate capacity estimates by a majority of participants are between 1,500 kips and 2,500 kips for any of the three directions. The capacity estimates of participants A, J, and H are above this range. Participant H used increased soil shear strength in its analysis which could be the reasons for higher capacity estimates.

The initial stiffness (linear part) indicates that the difference among the majority of participants varies within 40 percent. The initial stiffness of participants M and A are about 83 percent and 167 percent higher respectively, for the three directions.

Participant A and K subsequently provided revised load-displacement behavior which are included in Appendix A.

3.5 ULTIMATE STRENGTH ANALYSIS RESULTS (VOLUNTARY)

Six participants (B, C, D, J, K, M) provided ultimate strength analysis results, on a voluntary basis, for a "Fixed Base" case. Several participants performed analysis not defined in the required or voluntary portions of the Benchmark Basis Document and provided their results to the project. Their results are discussed in this Section.

3.5.1 Fixed Base Case

The results for the Fixed base case for three wave approach directions are summarized in Table 3-7. The ultimate capacity estimates per the participants are shown in Figure 3-13. The results for the three directions are discussed below.

- Direction 1: Four participants performed analysis for this direction and estimated ultimate capacity varying from 3,270 kips to 4,200 kips. The load level at first member with nonlinear event varied more significantly from 2,000 kips to 4,200 kips. Three participants noted leg yielding to govern failure of the platform.

- Direction 2: Six participants performed this analysis. The ultimate capacity estimate by participant J was significantly lower than those of other five participants. Participant B reported strut buckling as the governing failure mechanism, whereas all other 5 reported leg yielding to govern ultimate capacity estimate. The load level at first member failure varied from 1,100 kips to 4,060 kips. The RSR estimate varied significantly.

- Direction 3: The variation in ultimate capacity presented by five participants was lower for this direction compared to that for the other two directions. Participant B reported brace buckling to govern ultimate capacity, whereas three other participants noted leg yielding to govern as for other directions. The lateral load level when the first member experiences a nonlinear event was much lower for participant M than for other participants.

The following observations are made from comparison of the fixed base case results with those including soil effects (Section 3.4):

- For the two orthogonal directions (Direction 1 and Direction 3), the mean capacity estimates for the fixed base case are higher by 48 percent. The corresponding estimates of the standard deviations are lower by 30 and 50 percent and the resulting COV's half and one-third of those for the results for the cases with soil effect included.

- For the diagonal direction (Direction 2), the average mean capacity estimate for the fixed base case is significantly higher at 86 percent. The standard deviation of capacity estimate for the fixed base case for the diagonal direction increases by 68 percent, whereas it decreased for the two orthogonal directions. The decrease in COV is moderate for the fixed base case for this direction.

  Load-Displacement Behavior: Figures 3-14 to 3-16 compare the load-displacement behavior for the fixed base cases. A significant variation is noted in the initial stiffness (linear part) and post-failure behavior results provided by participants. The figures indicate two distinct stiffness bands of behaviors with M defining the lower bound and K defining the upper bound in all loading directions. The initial stiffness variation is within 33 percent for the three directions for participants D, J, and M. These represent a lower band for the stiffness estimates. The variation in stiffness compared to that of the lower bound stiffness (M) is between 120 to 160 percent for B, C, and K results. These comprise an upper band for the stiffness.

  It is interesting to note that in case of the analysis cases with soil effect included (Section 3.4), B and C showed lower stiffnesses and M showed a higher stiffness, which is opposite to the behavior noted for the fixed base case. This may be due to differences in considering fixity effect in their models.

3.5.2 Linear Elastic Analysis

Participants D, E, G, and L performed linear elastic analysis, with factors of safety included or excluded, before initiating ultimate strength analysis. They used Section 17 Design Level and/or Ultimate Strength loading criteria.

Participant D found overstressing of none of the elements of the platform when subjected to Section 17 Design Level loads and noted that the soil capacity (compression) governs with factor of safety exceeding 1.5, per RP 2A.

Participant E performed analysis for the diagonal direction and found overstressing of K joints and pile sections, when subjected to Section 17 Ultimate Strength loads.

Participant G performed analysis for eight directions to predict expected failure modes. The piles and segments of the jacket legs were found to be overstressed per RP 2A in several of the approach directions, when subjected to the Section 17 Design Level loads. Participant G also performed linear analysis by removing all factors of safety and noted that piles had formed plastic hinges but none of the other members were overloaded, when subjected to Section 17 Ultimate Strength loads.

Participant L analysis indicated that the pile axial loads exceed the punch-through and pullout capacity of the soil, when subjected to the Section 17 Ultimate Strength loads.

3.5.3 Effect of Joint Capacity

Several participants investigated the effect of including or not including joint capacity in their computer models. Only a few participants considered joint effects in their ultimate capacity analysis models. Some of them found that the effect of joint capacity was minimal on the ultimate capacity of platform (with pile/soil base case), whereas participant F found that its effect was significant in defining ultimate capacity of the platform.

Participant K investigated effect of joint flexibility and noted that the joints have little influence on the ultimate strength once their mean capacity is properly taken into account. However, the participant states that the modeling of joints did impact the mode of failure and post-peak response.

Some participants discussed modeling aspects of joints, which are given in Section 4.3.

3.5.4 Load Level Estimates for Higher Return Periods

Participant H developed metocean parameter values for the 200 year and 500 year return period storm cases. The magnitudes of maximum wave height were estimated as 69.2 ft. and 78 ft. for the 200 year and 500 year return period cases respectively. The participant reported maximum wave heights as 63 ft. and 68 ft. for 100 year return period storm and Section 17 ultimate strength criteria cases. The increase in wave-in-deck loads were significant for higher return period cases. For the 500 year return period case, the wave-in-deck loads varied from 30 % to 50 % of the load on jacket for three approach directions. For Direction 2, the total loads were reported as 2,318 kips, 3,209 kips and 5,002 kips for the 100 year, 200 year and 500 year return period cases respectively.

The participant found that the ultimate strength could vary significantly depending on how the pushover load is incremented from the 100-year loads to the ultimate failure. In addition, due to these loads becoming an increasing component of the total base shear for the higher return periods, further validation and calibration of the wave impact algorithm are important issues.

3.6 SUMMARY OF RESULTS

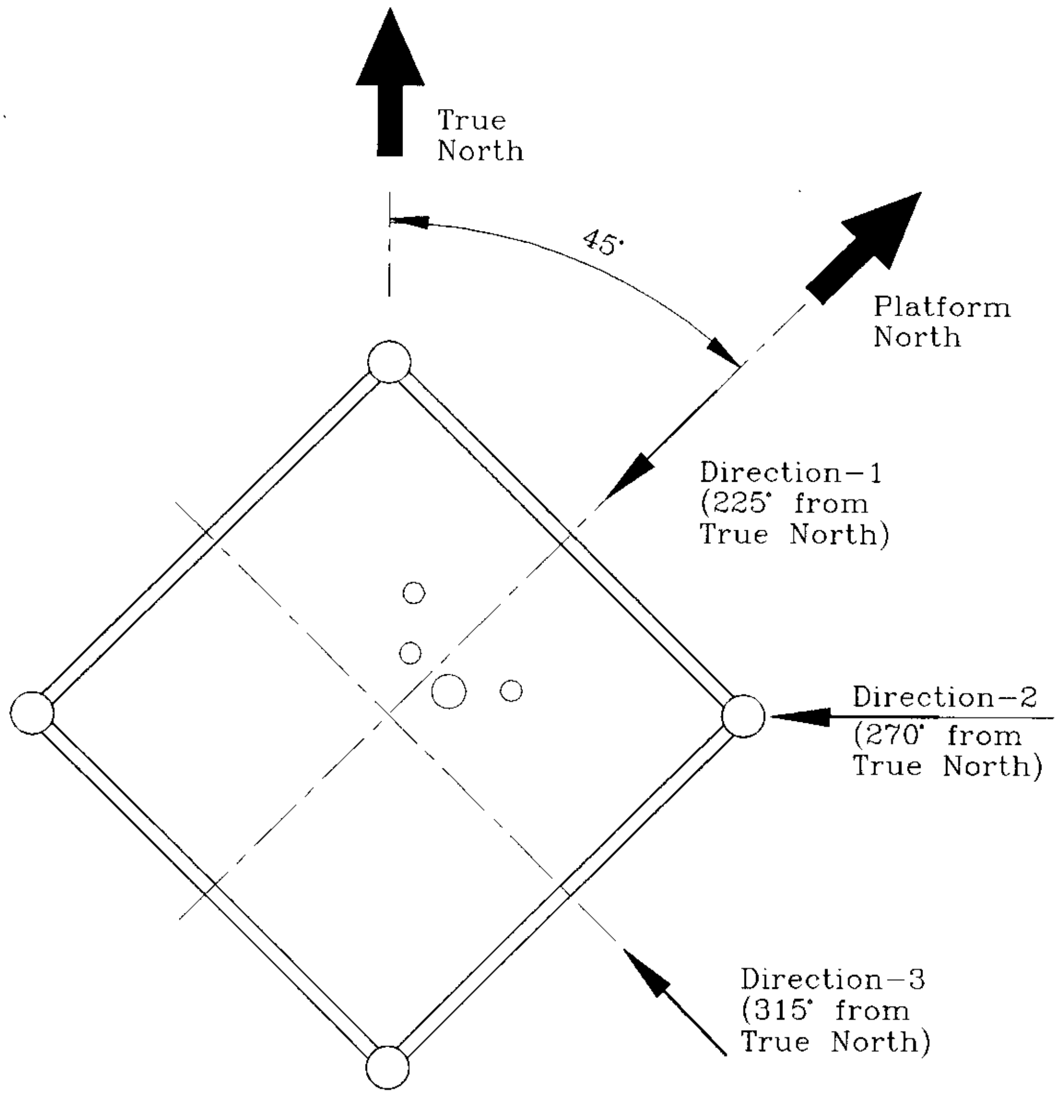
Table 3-8 and Figures 3-17 to 3-19 summarize variations in the metocean parameters, base shear, ultimate capacity, and RSR values for the three approach directions for the base case with pile/soil interaction included. These figures indicate significant variations in values obtained by the participants. Note that values for all parameters were not made available by all participants. Therefore, the range of values, mean, and COVs are based on available information, which is limited in some cases.

Figure 3-9 presented comparison of failure modes and mechanisms. A significant variation was noted among participants.

Based on the results for Direction 2, Figure 3-20 was developed to more clearly differentiate the results obtained by participants. In this figure, a subjective classification is attempted for wave height and base shear per RP 2A 20th Edition, and ultimate capacity of the platform. These quantities are classified as very low (VL), low (L), medium (M), high (H), very high (VH) on an assumed range of values for comparison purposes only. A single value is noted for the "Medium" wave height as it was used by most participants.

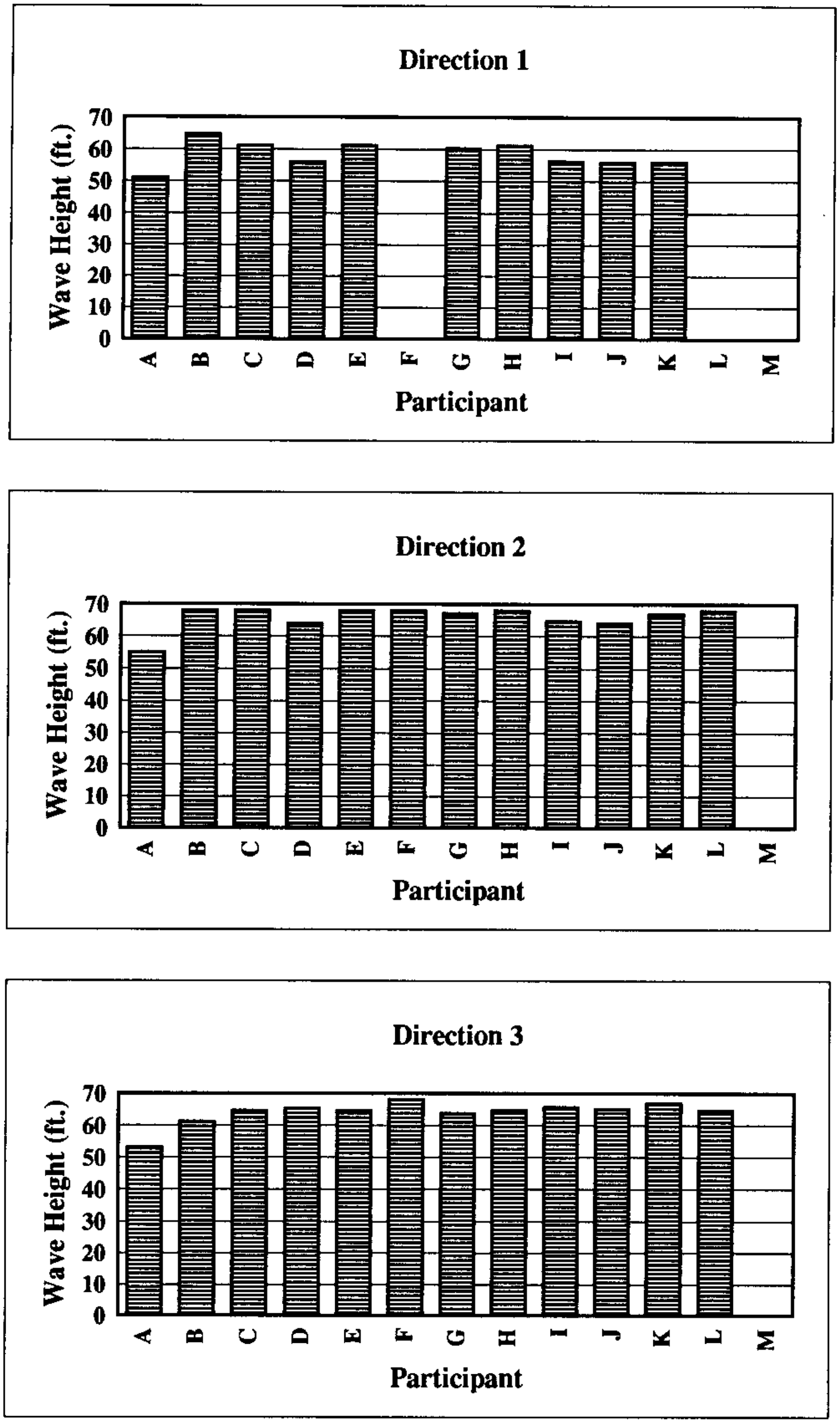
Figure 3-20(a) indicates VL (< 1,500 kips) base shear values estimated by participants A and D and VH (>3,000 kips) values by participant M. Participant G values are represented as "High" and participant F values as "Low" in this figure. The values per the other seven participants are in the "Medium" range (2,001 kips to 2,500 kips), whereas there is variation in wave height values among them. Participant J did not provide values for one or both quantities, and hence is not compared.

Figure 3-20(b) presents the assumed ranges for base shear and ultimate capacity for classification purpose. Participant M's 20th Edition base shear estimate is VH and ultimate capacity is noted "High." The capacity estimates for Participants A and D are noted as "High" and "Medium", whereas the base shear is in the VL category. Participant B's ultimate capacity estimate is in the VL category, whereas the base shear value is categorized as "Medium." Participant K did not perform analysis for this direction and participant J did not provide reference level base shear values, and were therefore excluded from the figure. Participant J ultimate capacity estimate is VH compared to others shown in the figure.



NOTE: The above three directions are basic directions referred to in the tables and figures. Tables 3−1 to 3−3 indicate normalized directions (with respect to True North) used in participants submittals.

Figure 3-1 Wave Approach Directions

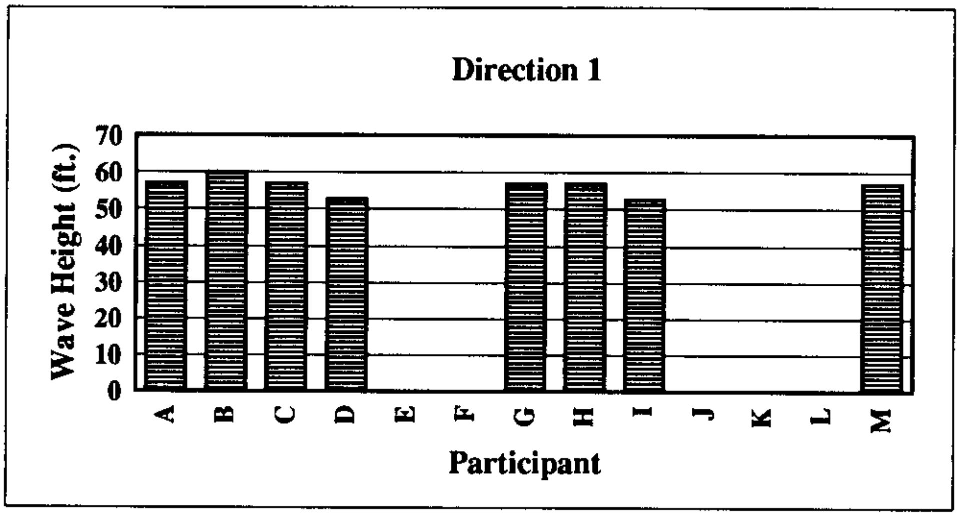


Mean	58.31
COV	0.07

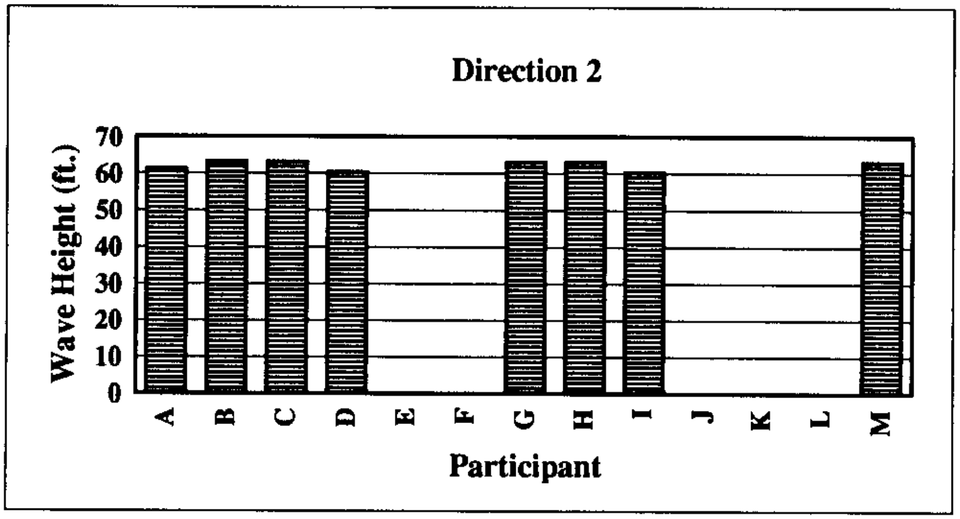
Mean	65.77
COV	0.06

Mean	63.92
COV	0.06

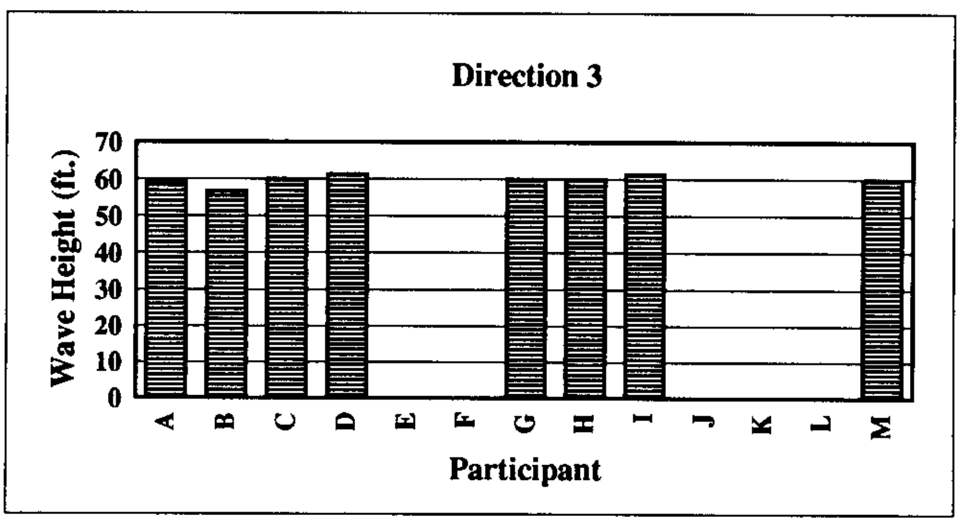
Figure 3-2: Comparison of Wave Height - Section 17 Ultimate Strength



Mean	56.04
COV	0.04

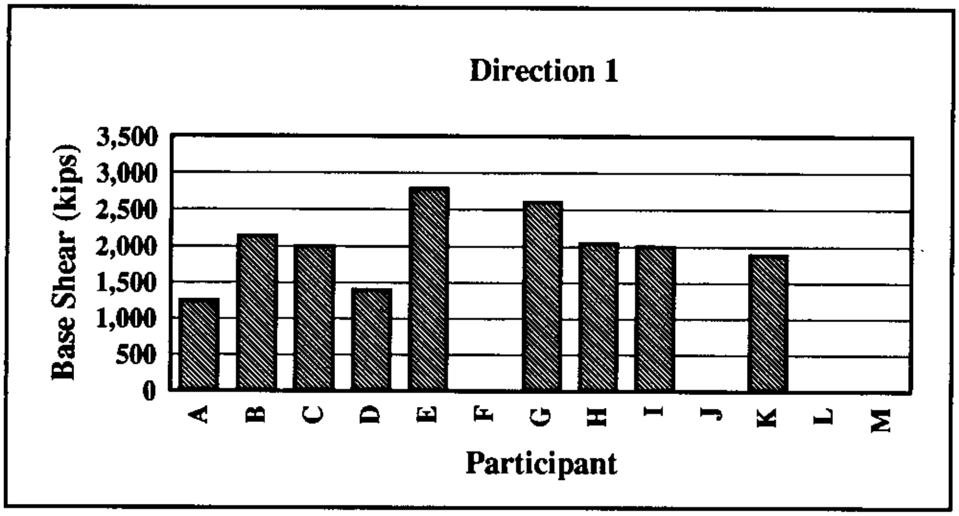


Mean	62.07
COV	0.02

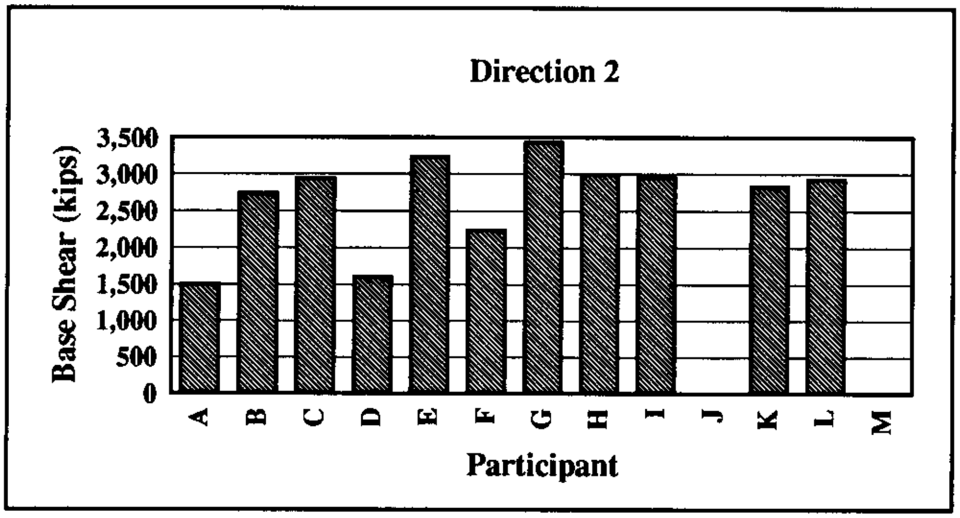


Mean	59.74
COV	0.02

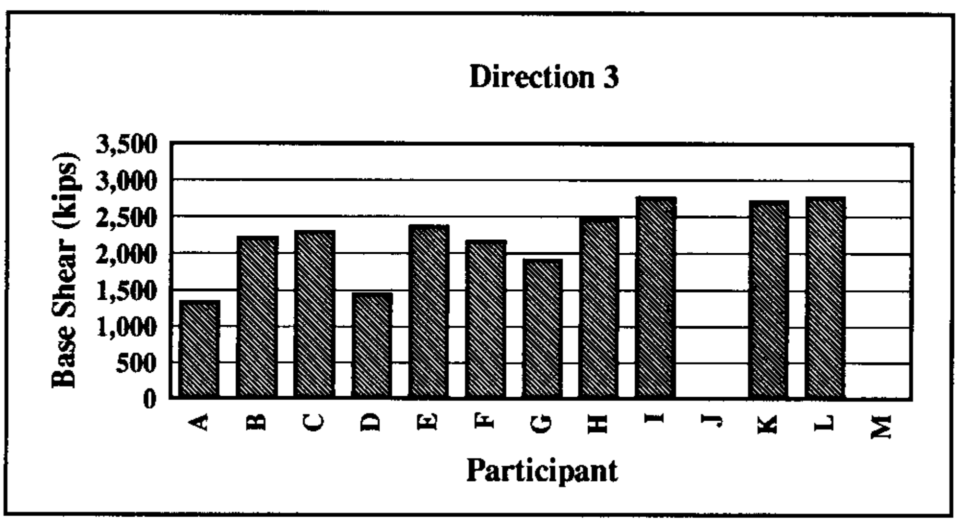
Figure 3-3: Comparison of Wave Height - RP 2A, 20th Edition



Mean	2,007
COV	0.25

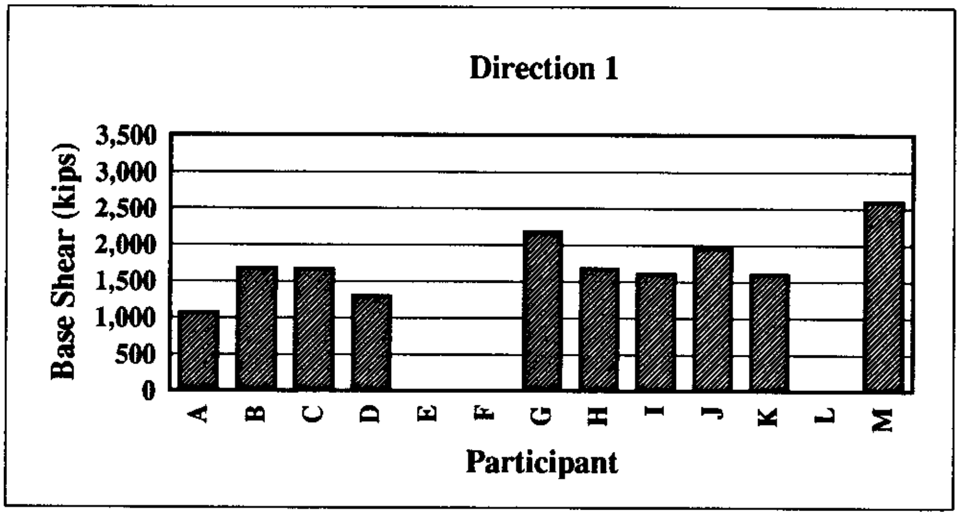


Mean	2,667
COV	0.24

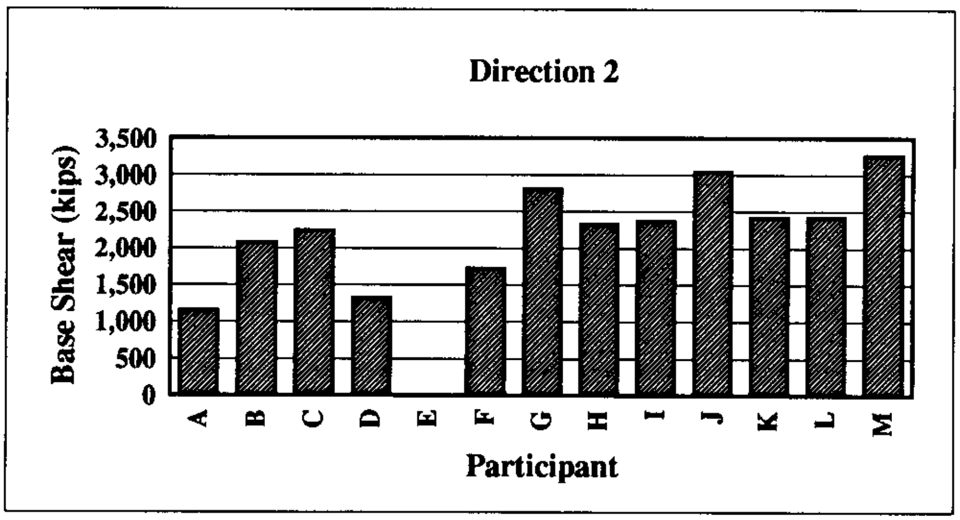


Mean	2,215
COV	0.22

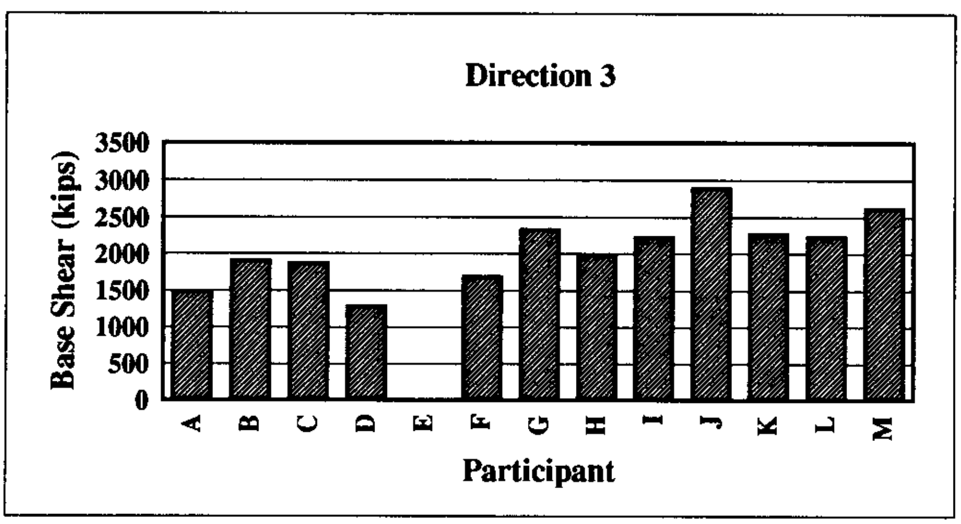
Figure 3-4: Comparison of Base Shear - Section 17 Ultimate Strength



Mean	1,725
COV	0.25

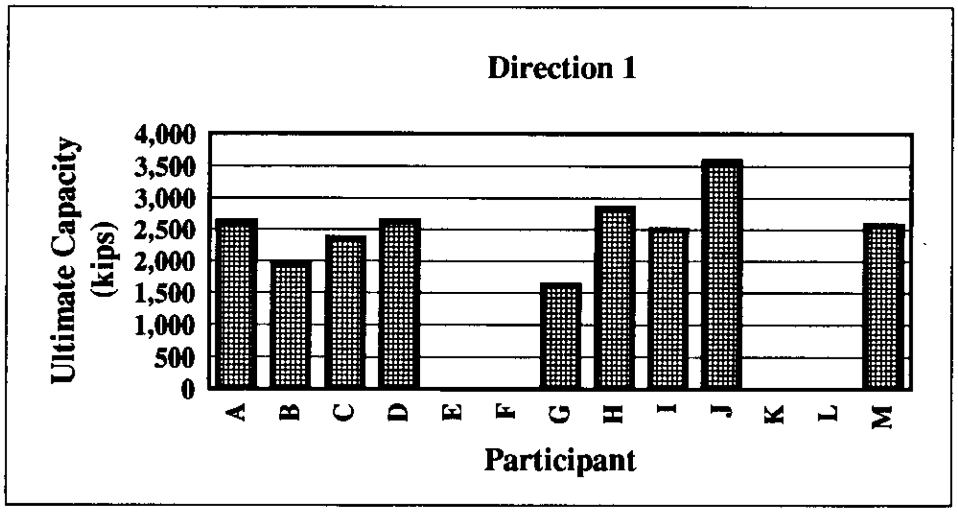


Mean	2,260
COV	0.28

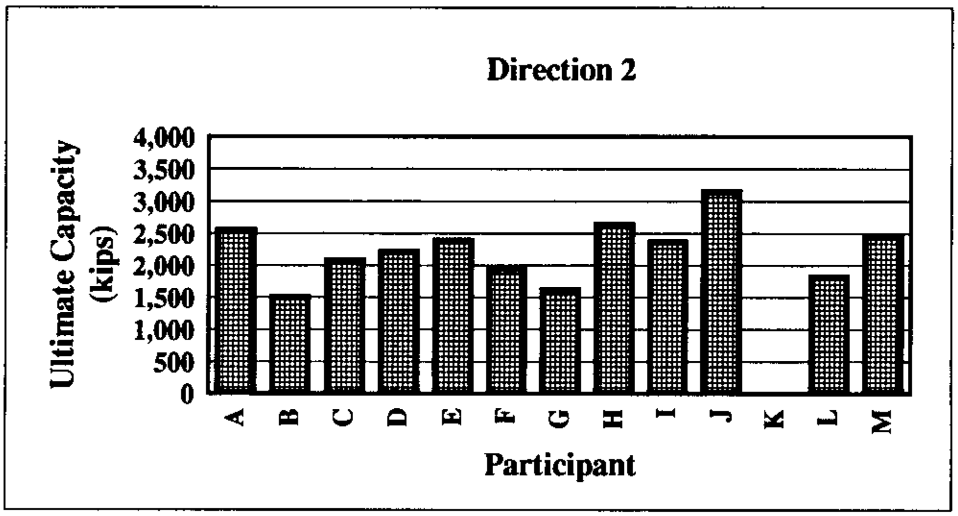


Mean	2,061
COV	0.22

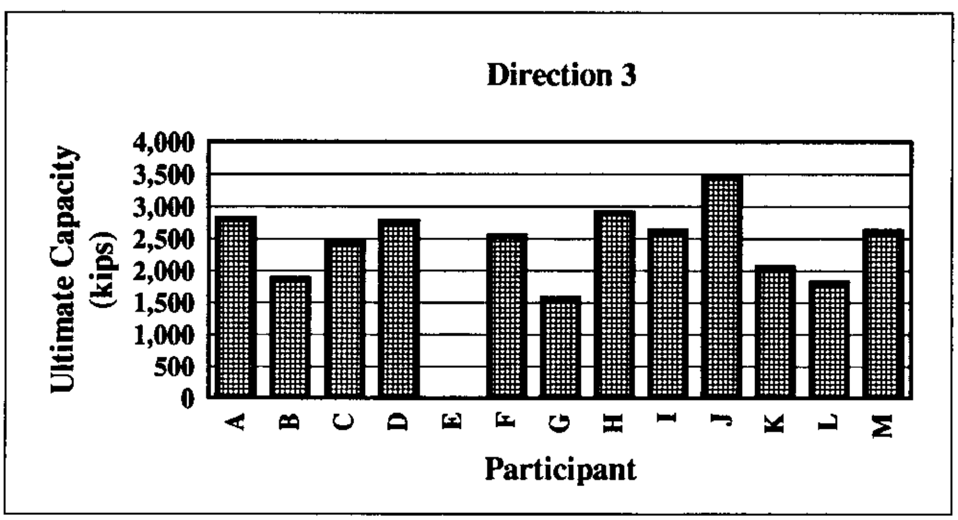
Figure 3-5: Comparison of Base Shear - RP 2A, 20th Edition



Mean	2,513
COV	0.22

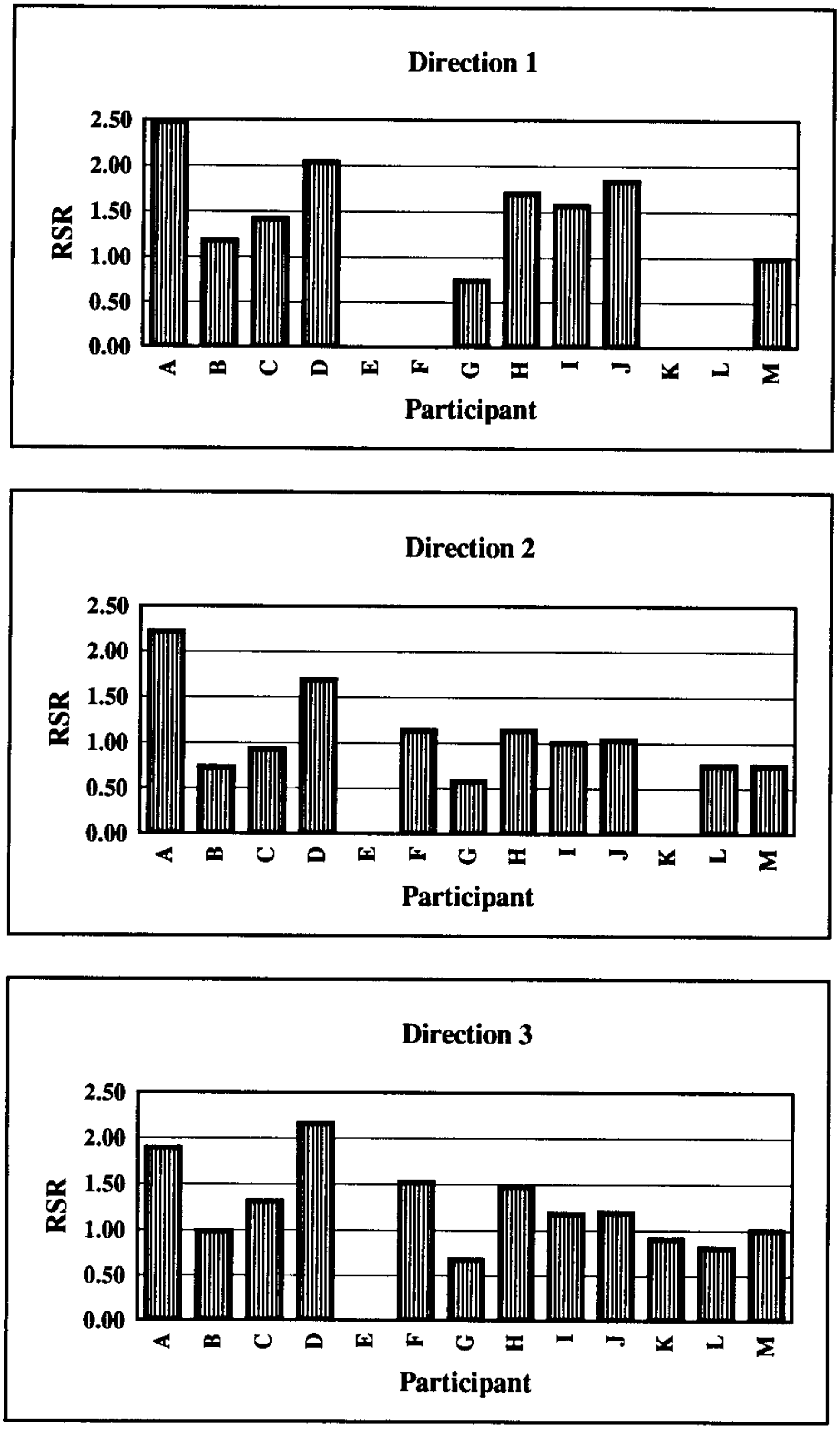


Mean	2,219
COV	0.21



Mean	2,446
COV	0.22

Figure 3-6: Comparison of Ultimate Capacity (Ru)



Mean	1.55
COV	0.35

Mean	1.08
COV	0.44

Mean	1.25
COV	0.35

Figure 3-7: Comparison of Reserve Strength Ratio (RSR)

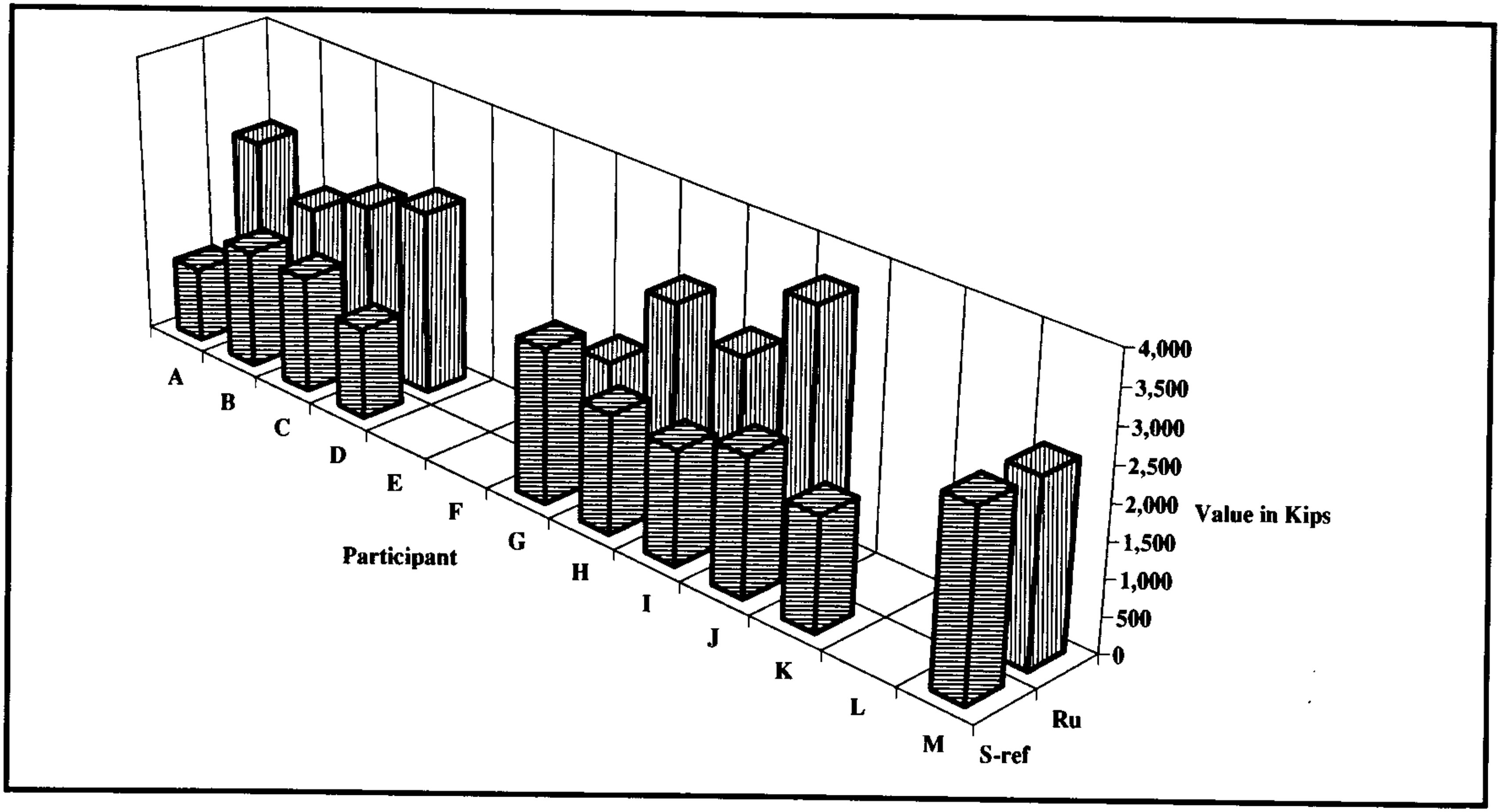


Figure 3-8(a): Variation of Reference Level Load and Ultimate Capacity - Direction 1

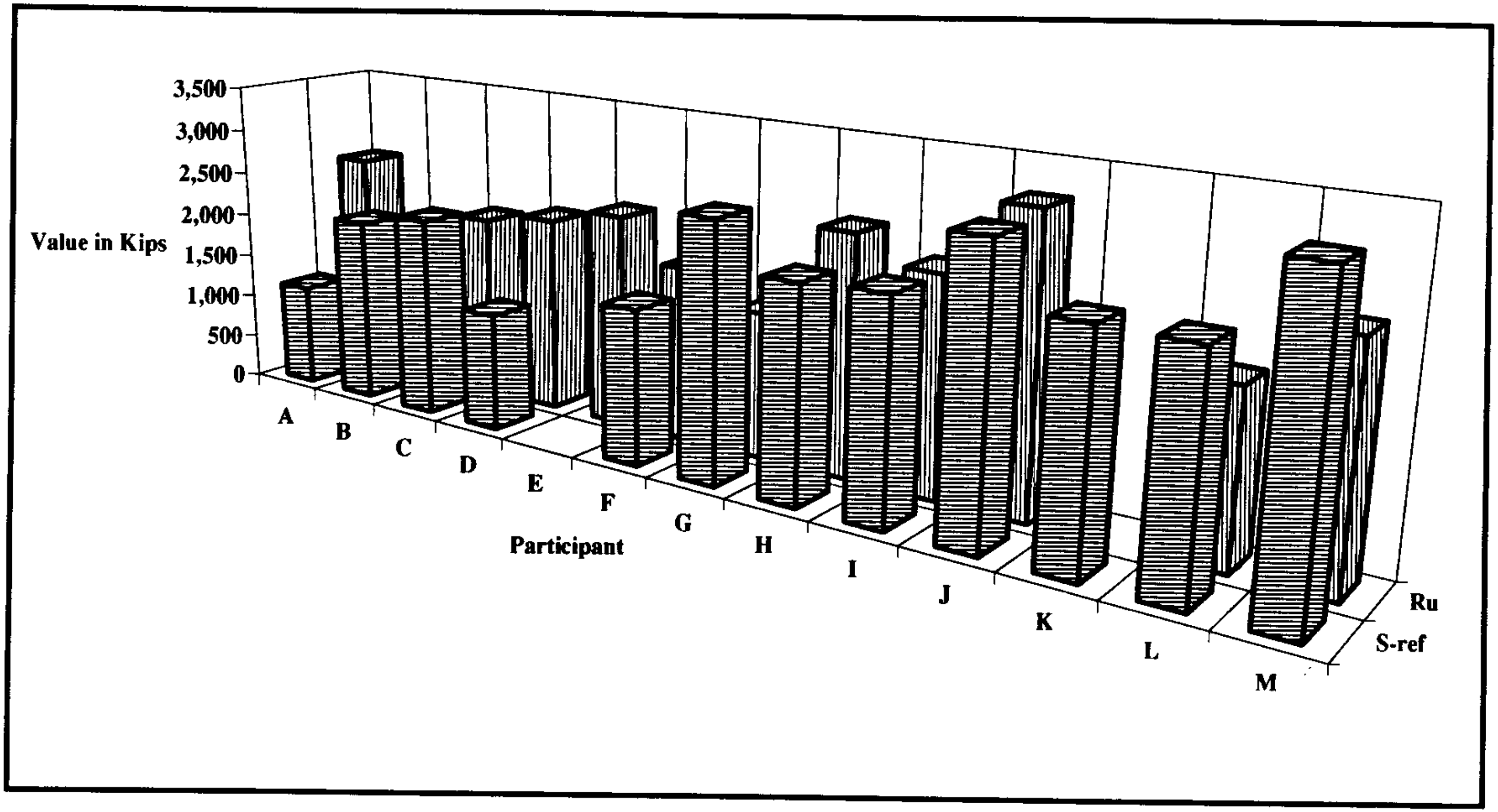


Figure 3-8(b): Variation of Reference Level Load and Ultimate Capacity - Direction 2

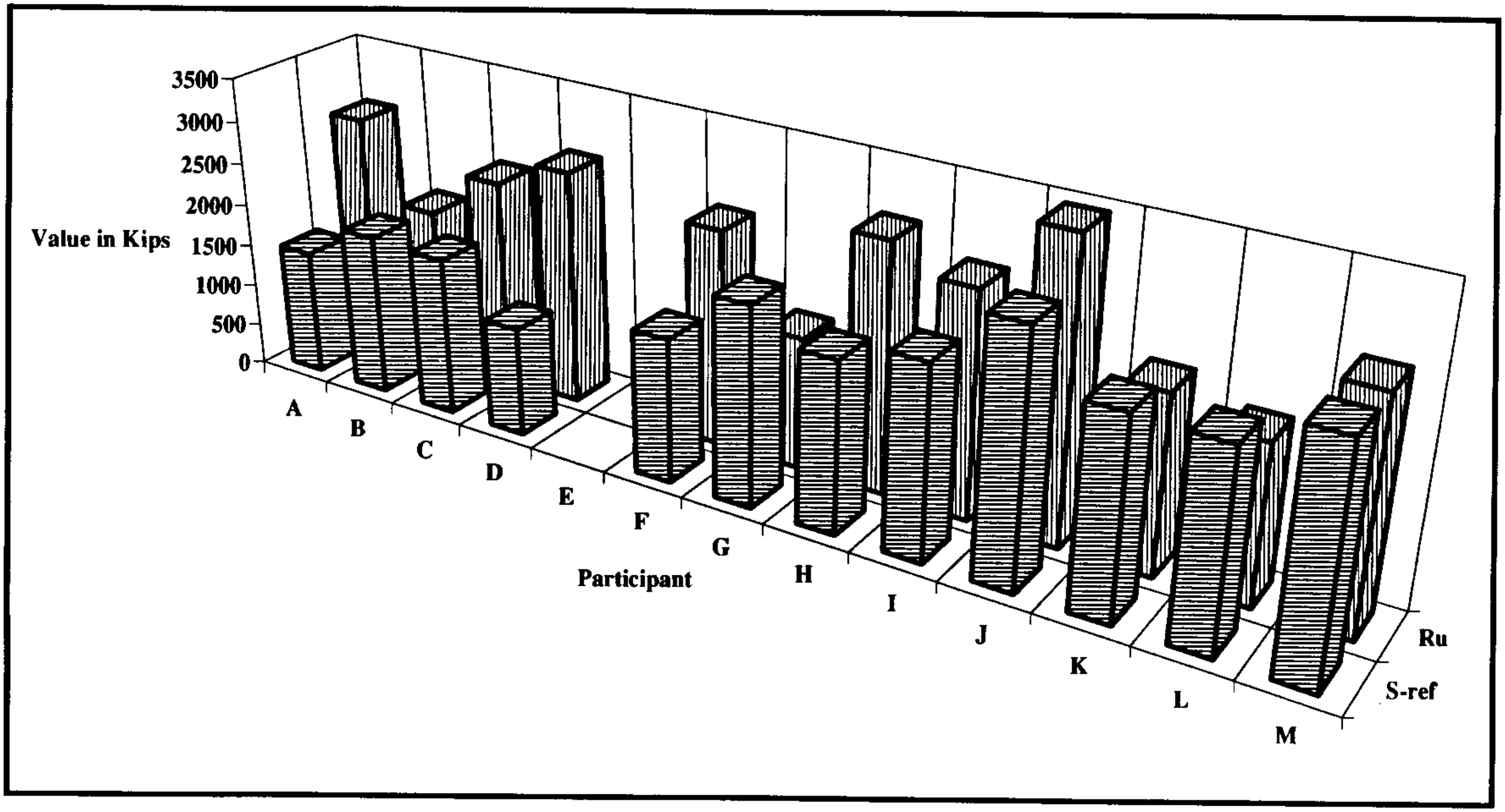


Figure 3-8(c): Variation of Reference Level Load and Ultimate Capacity - Direction 3

Component Failure Modes	Participant												
	A	B	C	D	E	F	G	H	I	J	K	L	M
Jacket Components													
Leg Yielding		Yes	Yes	Yes	Yes			Yes		Yes			
K-brace										Yes			
K-joint					Yes								
Horizontals					Yes			Yes	Yes			Yes	
Pile Sections:													
First Yielding	Yes	Yes						Yes				Yes	
Full Yielding		Yes											
Double Hinging	Yes					Yes	Yes						
Soil Capacity:													
Compression Capacity	Yes		Yes	Yes	Yes		Yes		Yes			Yes	Yes
Tensile Capacity													

Platform Failure Mode (#1)	Soil Capacity	Pile Yielding	Pile Plunging	Foundation (Soil Capacity)	Soil Capacity	Foundation/ Pile double hinging	Pile/ Soil	Pile Yielding	Jacket	Legs & Braces		Jacket-Pile	Soil

Note #1: Per Participants' Submittals

Figure 3-9: Comparison of Component and Platform Failure Modes - Direction 2 (270 degree from True North) - Revised

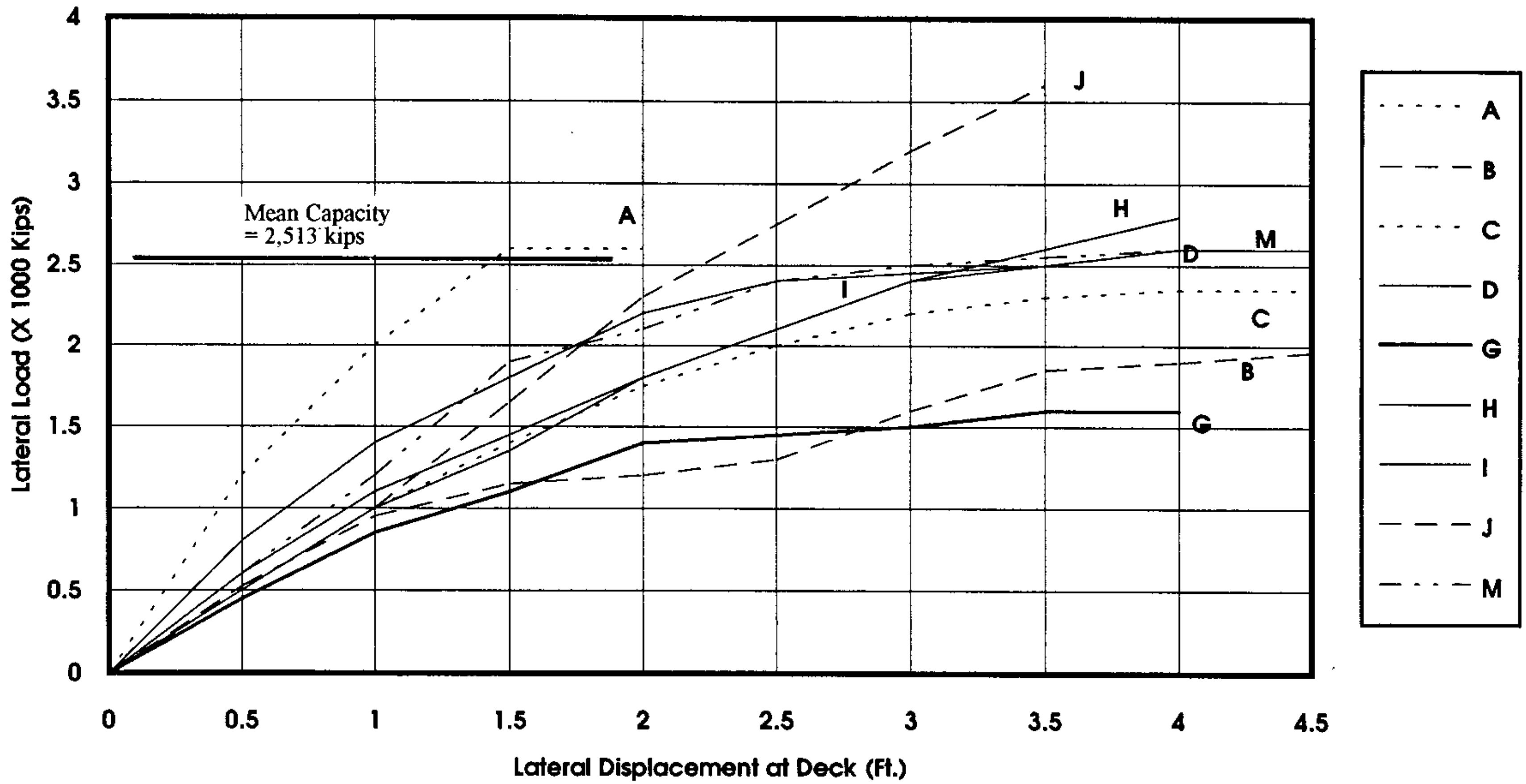


Figure 3-10 Load-Displacement Behavior - Direction 1

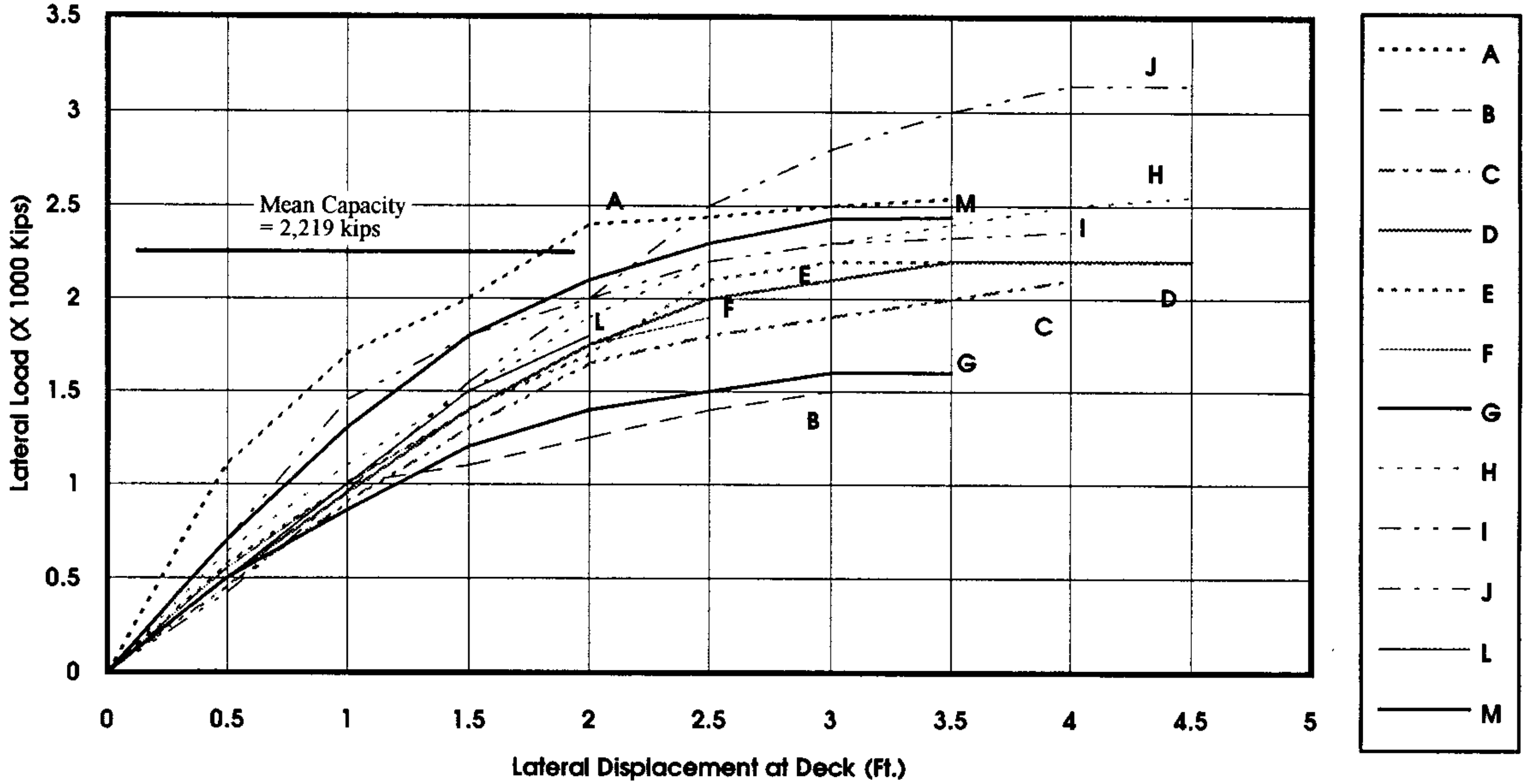


Figure 3-11 Load-Displacement Behavior - Direction 2

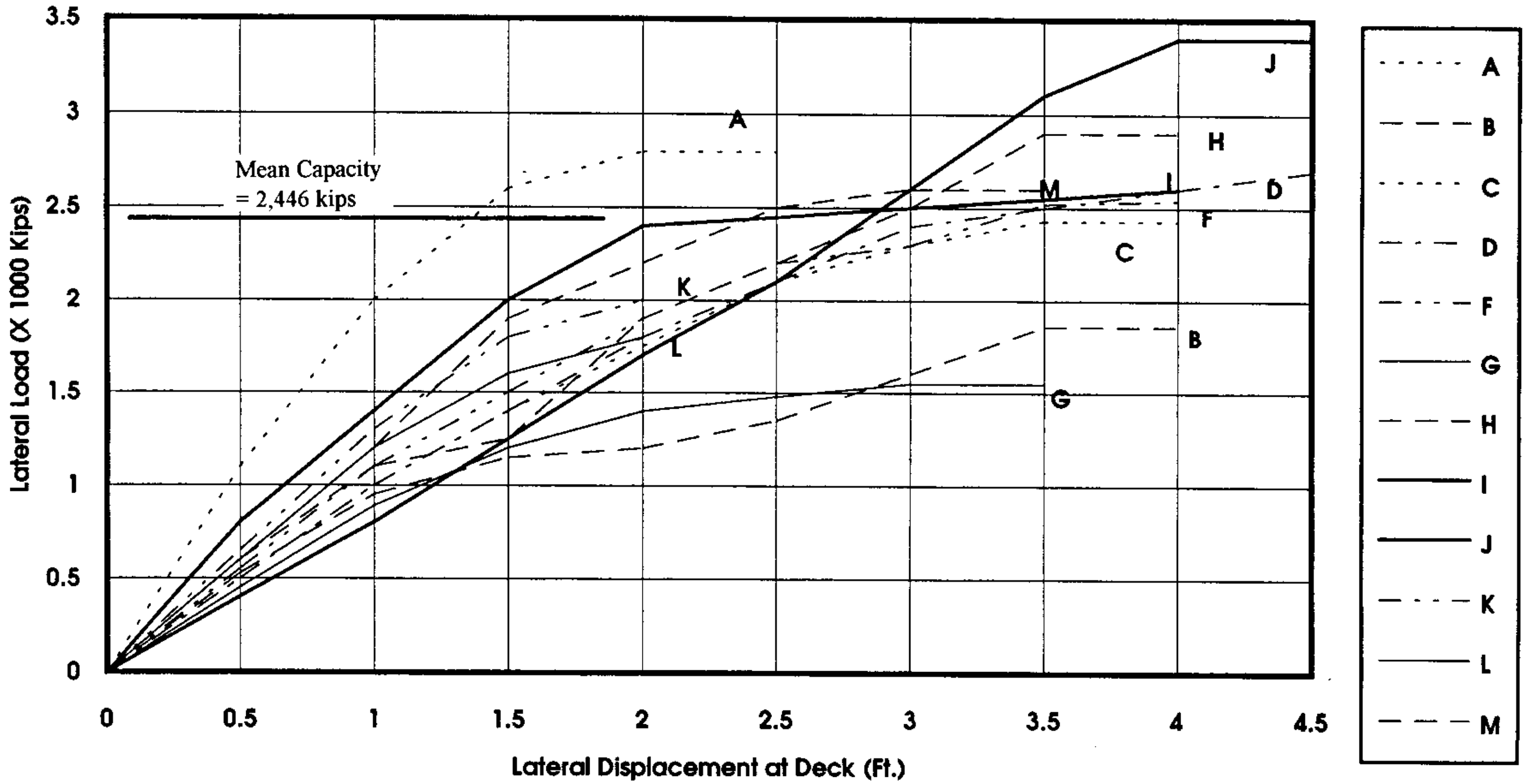
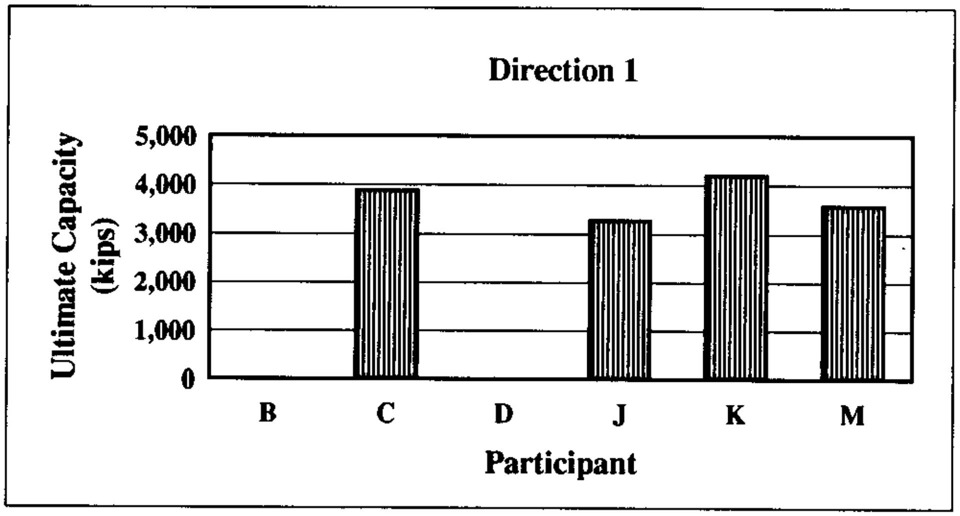
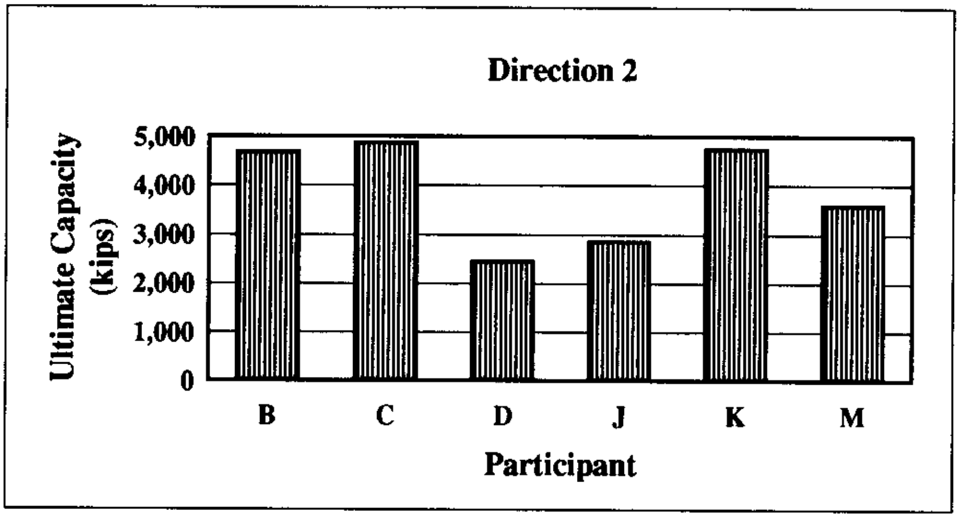


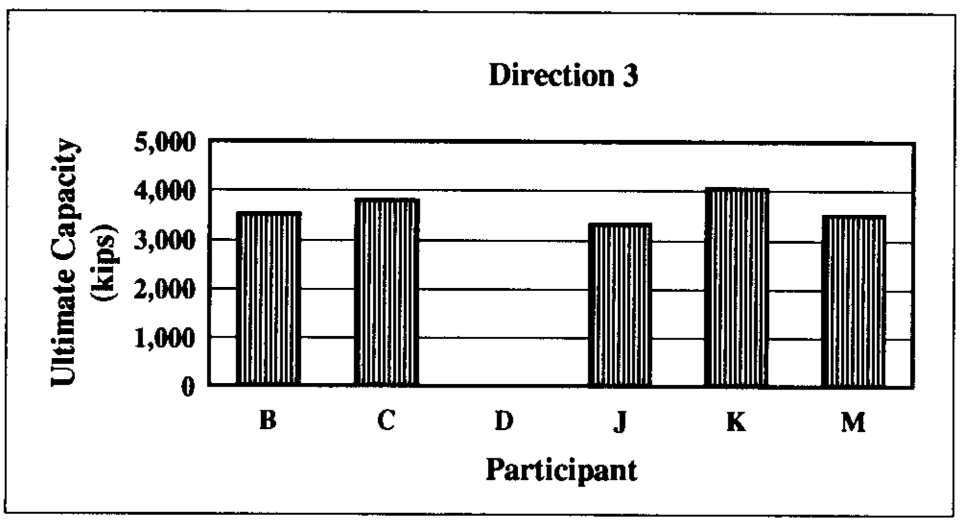
Figure 3-12 Load-Displacement Behavior - Direction 3



Mean	3,726
COV	0.11



Mean	3,862
COV	0.27



Mean	3,642
COV	0.08

Figure 3-13: Comparison of Ultimate Capacity (Ru) - Fixed base Case

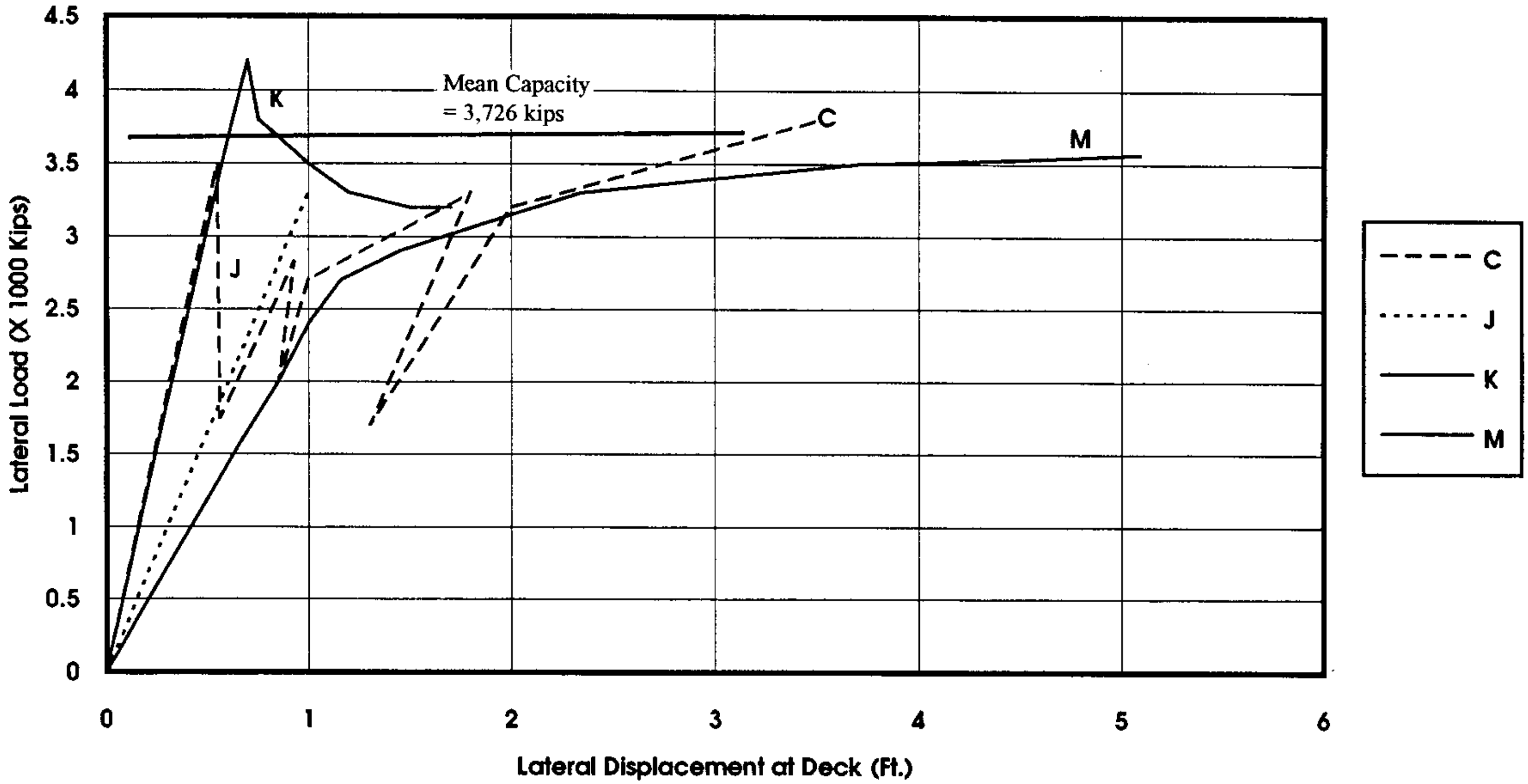


Figure 3-14 Load-Displacement Behavior - Fixed Base Case - Direction 1

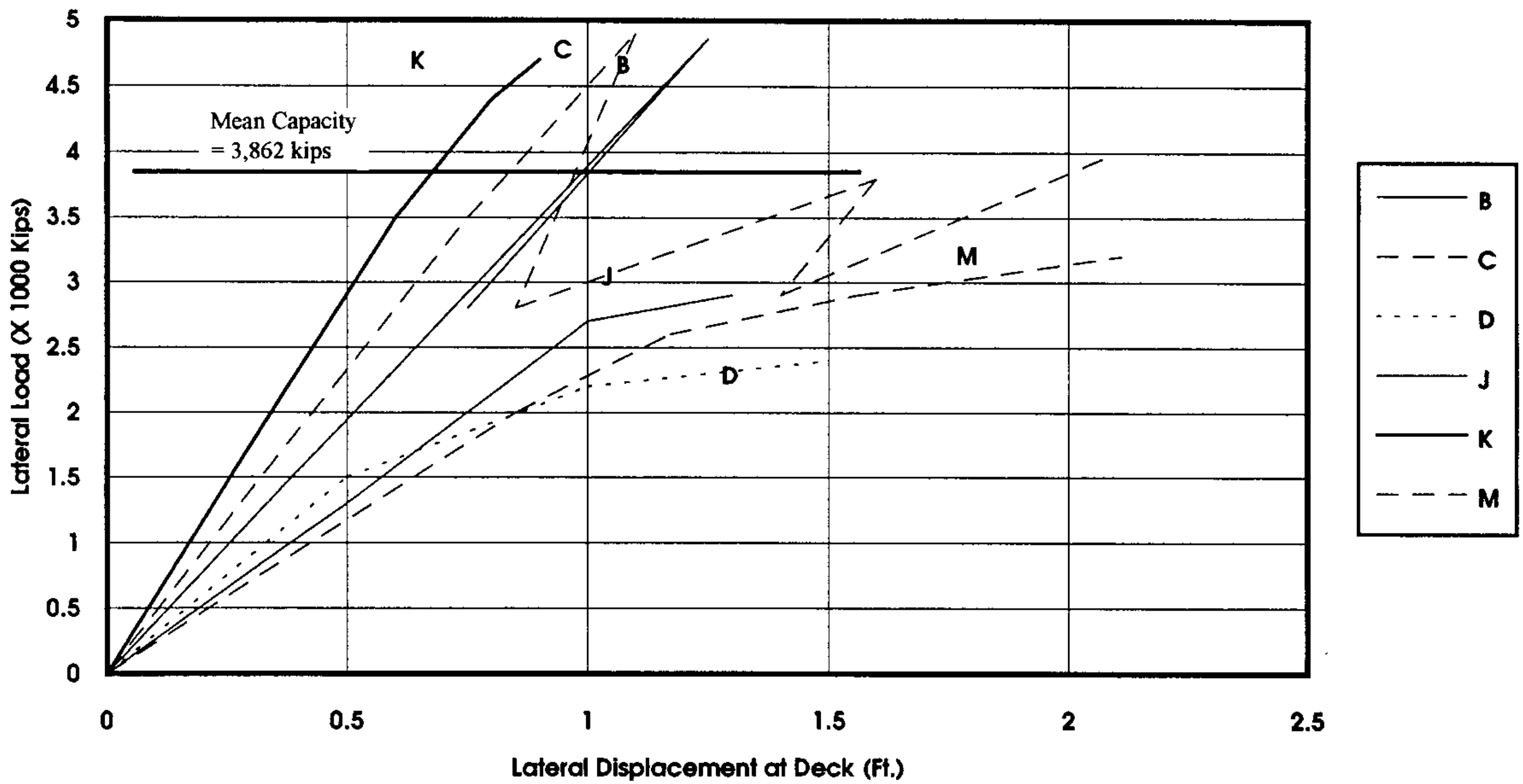


Figure 3-15 Load-Displacement Behavior - Fixed Base Case - Direction 2

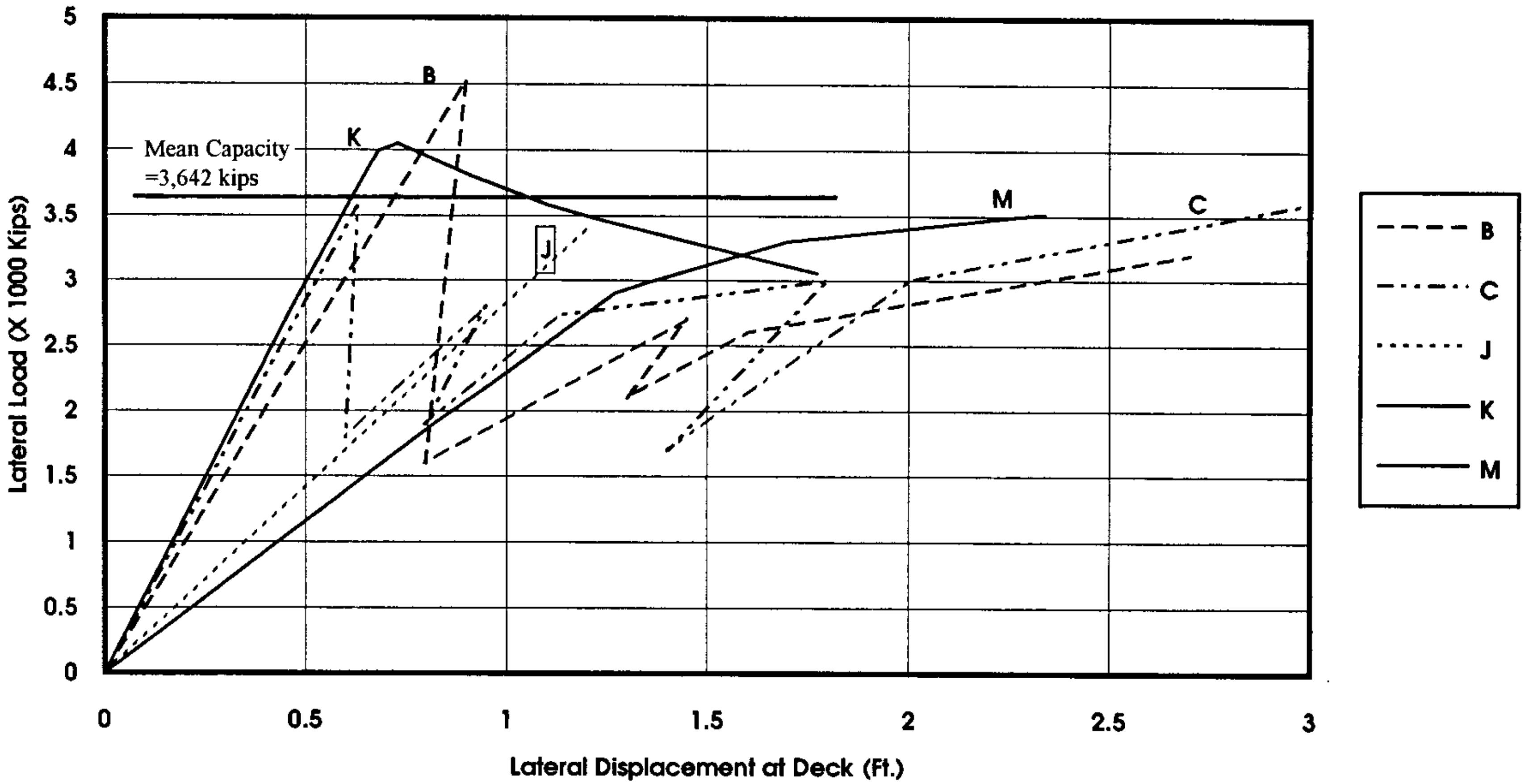


Figure 3-16 Load-Displacement Behavior - Fixed Base Case - Direction 3

		Mean	Cov (%)	Per API TG
Wave Height Section 17 (Ult. Str.)	50.90 – 64.60	58.31	7	61.2
Wave Height RP2A, 20th Ed.	51.72 – 59.85	55.68	5	56.7

a) **Metocean Criteria - Wave Height (ft)**

In-Line Current Speed Section 17 (Ult. Str.)	1.7 – 3.88	2.68	28	2.31
In-Line Current Speed RP2A, 20th Ed.	1.56 – 3.54	2.57	28	2.11

b) **Metocean Criteria - Current Speed (ft/sec)**

Base Shear Section 17 (Ult. Str.)	1,243 – 2,780	2,001	23
Base Shear RP2A, 20th Ed. (S_{ref})	1,056 – 2,590	1,735	25
Load @ First Member with NLinear Event	1,119 – 3,527	1,831	41
Ultimate Capacity (R_u)	1,610 – 3,573	2,513	22

c) **Analysis Results - Load Levels (kips)**

Reserve Strength Ratio (RSR)	0.74 – 2.47	1.51	37

d) **Analysis Results - RSR**

Figure 3-17 Summary of Variations of Metocean Parameters and Analysis Results - Direction-1 (Revised)

.046\TRIALS\FIG-3-10

	Range (low)	Range (high)	Mean	Cov (%)	Per API TG
Wave Height Section 17 (Ult. Str.)	54.80	68.00	65.77	6	68
Wave Height RP2A, 20th Ed.	60.20	63.00	62.29	2	63

a) **Metocean Criteria - Wave Height (ft)**

	Range (low)	Range (high)	Mean	Cov (%)	Per API TG
In-Line Current Speed Section 17 (Ult. Str.)	3.00	3.88	3.69	8	3.83
In-Line Current Speed RP2A, 20th Ed.	3.02	3.54	3.44	6	3.49

b) **Metocean Criteria - Current Speed (ft/sec)**

	Range (low)	Range (high)	Mean	Cov (%)
Base Shear Section 17 (Ult. Str.)	1,495	3,429	2,699	23
Base Shear RP2A, 20th Ed. (S_{ref})	1,150	3,265	2,210	27
Load @ First Member with NLinear Event	980	2,295	1,616	26
Ultimate Capacity (R_u)	1,496	3,143	2,209	22

c) **Analysis Results - Load Levels (kips)**

	Range (low)	Range (high)	Mean	Cov (%)
Reserve Strength Ratio (RSR)	0.57	2.21	1.07	45

d) **Analysis Results - RSR**

Figure 3-18 Summary of Variations of Metocean Parameters and Analysis Results - Direction-2 (Revised)

.046\TRIALS\FIG-3-11

	Range	Mean	Cov(%)	Per API TG
Wave Height Section 17 (Ult. Str.)	53.10 – 68.00	63.92	6	64.6
Wave Height RP2A, 20th Ed.	56.70 – 63.00	60.22	3	59.9

a) **Metocean Criteria - Wave Height (ft)**

	Range	Mean	Cov(%)	Per API TG
In-Line Current Speed Section 17 (Ult. Str.)	3.00 – 3.88	3.41	11	3.21
In-Line Current Speed RP2A, 20th Ed.	2.75 – 3.54	3.22	11	2.94

b) **Metocean Criteria - Current Speed (ft/sec)**

	Range	Mean	Cov(%)
Base Shear Section 17 (Ult. Str.)	1,331 – 2,884	2,271	22
Base Shear RP2A, 20th Ed. (S_{ref})	1,280 – 2,613	2,008	19
Load @ First Member with NLinear Event	1,060 – 3,417	1,881	36
Ultimate Capacity (R_u)	1,550 – 3,439	2,446	22

c) **Analysis Results - Load Levels (kips)**

	Range	Mean	Cov(%)
Reserve Strength Ratio (RSR)	0.67 – 2.16	1.26	37

d) **Analysis Results - RSR**

Figure 3-19 Summary of Variations of Metocean Parameters and Analysis Results - Direction-3 (Revised)

.046\TRIALS\FIG-3-12

Wave Height (ft.)			< 1,500 (V. Low)	1,501-2,000 (Low)	2,001-2,500 (Medium)	2,501-3,000 (High)	> 3,000 (V. High)
	V. High	> 65					
	High	63.1 - 65					
	Medium	63		F	B,C,E,H,L	G	M
	Low	60 - 62.9	A, D		I, K		
	V. Low	< 60					

20th Edition Base Shear (Kips)

Participant J did not provide sufficient information (Ref. Table 3-2)
to be included in this chart

a) Based on Selected Wave Height (20th Edition) and Base Shear

Ultimate Capacity (Kips)			< 1,500 (V. Low)	1,501-2,000 (Low)	2,001-2,500 (Medium)	2,501-3,000 (High)	> 3,000 (V. High)
	V. High	> 2,700					
	High	2,301-2,700	A		E, H, I		M
	Medium	1,901-2,300	D	F	C		
	Low	1,501-1,900			L	G	
	V. Low	< 1,500			B		

20th Edition Base Shear (Kips)

Participant J did not provide sufficient information (Ref. Table 3-5) and
participant K did not perform analysis for Direction-2 to be included in this chart

b) Based on Reference Level Base Shear and Ultimate Capacity

Fig. 3-20: Classification Based on Wave Height and Analysis Results-Direction 2(Rev.)

Table 3-1: Comparison of Metocean Parameters and Loads: Direction 1 (225 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	In-Line Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	In-Line Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	45	50.90	3.00	1,243	0	1,243	56.70	3.50	1,056	0	1,056
B	225	64.60	1.94	1,821	319	2,140	59.85	1.77	1,545	125	1,670
C	225	61.20	2.40	1,895	100	1,995	56.70	2.19	1,570	90	1,660
D	225	55.80	2.50	1,352	38	1,390	52.50	2.28	1,273	17	1,290
E	225	61.20	1.70	2,732	48	2,780	56.70	1.56	2,028	23	2,051
F	-	-	-	-	-	-	-	-	-	-	-
G (#3)	225	60.30	3.88	2,604	0	2,604	56.70	3.54	2,174	0	2,174
H (#2)	225	61.20	3.11	1,860	95	2,038	56.70	2.84	1,535	60	1,669
I (#2)	225	56.23	2.44	1,940	0	1,992	52.48	2.23	1,600	0	1,600
J	225	55.80	2.06	1,948	0	1,948	-	-	-	-	-
K (#4)	225	55.83	3.76	1,840	40	1,880	51.72	2.32	1,380	210	1,590
L	-	-	-	-	-	-	-	-	-	-	-
M	245	-	-	-	-	-	56.70	3.50	2,525	65	2,590
Mean		58.31	2.68	1,924	64	2,001	55.68	2.57	1,669	59	1,735
St. Dev.		4.04	0.74	463	97	466	2.57	0.73	445	68	439
COV		0.07	0.28	0.24	1.52	0.23	0.05	0.28	0.27	1.15	0.25

Notes:
#1: The gray boxes identify the significantly different value or the lower and upper values for columns.
#2: Total base shear includes wind load on the deck
#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.
#4: Considered wave-in-deck loads above El. (+) 16'.

Table 3-2: Comparison of Metocean Parameters and Loads: Direction 2 (270 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	360	54.80	3.00	1,495	0	1,495	61.10	3.50	1,150	0	1,150
B	270	68.00	3.36	2,241	486	2,727	63.00	3.07	1,952	118	2,070
C	270	68.00	3.86	2,580	355	2,935	63.00	3.52	2,065	165	2,230
D	270	64.00	3.87	1,492	98	1,590	60.20	3.54	1,247	63	1,310
E	270	68.00	3.67	3,152	78	3,230	63.00	3.37	2,380	71	2,451
F (#2)	270	68.00	3.88	1,783	364	2,234	63.00	3.54	1,481	155	1,713
G (#3)	270	67.00	3.88	3,330	99	3,429	63.00	3.54	2,810	0	2,810
H (#2)	270	68.00	3.31	2,568	340	2,992	63.00	3.02	2,134	110	2,318
I (#2)	270	64.53	3.86	2,872	0	2,951	60.23	3.52	2,367	0	2,367
J	270	64.02	3.80	3,050	0	3,050	-	-	-	-	-
K (#4)	270	66.84	3.87	2,632	198	2,830	61.93	3.52	1,994	426	2,420
L	270	68.00	3.88	2,782	140	2,922	63.00	3.54	2,296	123	2,419
M	290	-	-	-	-	-	63.00	3.54	3,137	128	3,265
Mean		65.77	3.69	2,498	180	2,699	62.29	3.44	2,084	113	2,210
St. Dev.		3.82	0.30	623	167	611	1.14	0.19	589	115	592
COV		0.06	0.08	0.25	0.93	0.23	0.02	0.06	0.28	1.02	0.27

Notes:
#1: The gray boxes identify the significantly different value or the lower and upper values for columns.
#2: Total base shear includes wind load on the deck
#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.
#4: Considered wave-in-deck loads above El. (+) 16'.

Table 3-3: Comparison of Metocean Parameters and Loads: Direction 3 (315 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	315	53.10	3.00	1,331	0	1,331	59.20	3.50	1,482	0	1,482
B	315	61.00	3.75	2,079	130	2,209	56.70	3.42	1,780	120	1,900
C	315	64.60	3.06	2,150	145	2,295	59.90	2.80	1,740	120	1,860
D	315	65.20	3.00	1,330	110	1,440	61.30	3.54	1,216	64	1,280
E	315	64.60	3.39	2,313	47	2,360	59.90	3.14	2,202	44	2,246
F (#2)	315	68.00	3.88	1,788	309	2,162	63.00	3.54	1,492	130	1,680
G (#3)	315	63.70	3.88	1,907	0	1,907	59.90	3.54	2,325	0	2,325
H (#2)	315	64.60	3.11	2,196	165	2,449	59.85	2.84	1,815	85	1,982
I (#2)	315	65.61	3.02	2,697	0	2,755	61.24	2.75	2,227	0	2,227
J	315	65.12	3.30	2,884	0	2,884	-	-	-	-	-
K (#4)	315	66.91	3.82	2,452	250	2,702	61.99	2.77	1,859	409	2,268
L	315	64.60	3.69	2,628	127	2,755	59.85	3.37	2,123	112	2,235
M	335	-	-	-	-	-	59.85	3.47	2,505	108	2,613
Mean		63.92	3.41	2,146	107	2,271	60.22	3.22	1,897	99	2,008
St. Dev.		3.80	0.37	498	103	504	1.57	0.34	388	110	386
COV		0.06	0.11	0.23	0.96	0.22	0.03	0.11	0.20	1.10	0.19

Notes:
#1: The gray boxes identify the significantly different value or the lower and upper values for columns.
#2: Total base shear includes wind load on the deck
#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.
#4: Considered wave-in-deck loads above El. (+) 16'.

Table 3-4: Ultimate Strength Analysis Results: Direction 1 (225 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A (#2)	1,243	1,056	-	1,119	2,612	2.47	-
B	2,140	1,670	641	1,186	1,964	1.18	First leg/ Pile yielding
C	1,995	1,660	-	1,990	2,350	1.42	Foundation
D	1,390	1,290	1,375	2,294	2,623	2.03	Foundation
E	2,780	2,051	-	-	-	-	-
F	-	-	-	-	-	-	-
G	2,604	2,174	-	1,317	1,610	0.74	Pile, Soil
H	2,038	1,669	1,260	1,630	2,827	1.69	Pile/horz. braces/deck leg
I	1,992	1,600	930	1,494	2,490	1.56	Jacket/Foundation
J	1,948	-	-	3,527	3,573	-	Legs and braces
K	1,880	1,590	-	-	-	-	-
L	-	-	-	-	-	-	-
M	-	2,590	-	1,921	2,564	0.99	Soil
Mean	2,001	1,735	1,052	1,831	2,513	1.51	
St. Dev.	466	439	332	747	547	0.56	
COV	0.23	0.25	0.32	0.41	0.22	0.37	

Notes:

#1: The gray boxes identify the lower and upper values for columns.

#2: Participants' RSR estimates differed

Table 3-5: Ultimate Strength Analysis Results: Direction 2 (270 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A (#2)	1,495	1,150	-	1,196	2,542	2.21	Pile hinge/Compression Piles
B	2,727	2,070	807	1,166	1,496	0.72	First leg/ Pile yielding
C	2,935	2,230	-	1,920	2,070	0.93	Foundation
D	1,590	1,310	1,500	1,990	2,200	1.68	Foundation
E (#2)	3,230	2,451	-	1,290	2,381	0.97	Soil capacity
F	2,234	1,713	-	1,937	1,937	1.13	Foundation/ pile double hinging
G	3,429	2,810	-	980	1,610	0.57	Pile, Soil
H	2,992	2,318	1,252	1,636	2,628	1.13	Pile, leg, horiz. braces
I	2,951	2,367	785	1,770	2,361	1.00	Jacket/Foundation
J	3,050	-	-	2,295	3,143	-	Legs and braces
K	2,830	2,420	-	-	-	-	-
L	2,922	2,419	-	1,315	1,689	0.70	Jacket, Pile
M	-	3,265	-	1,900	2,446	0.75	Soil
Mean	2,699	2,210	1,086	1,616	2,209	1.07	
St. Dev.	611	592	350	414	477	0.48	
COV	0.23	0.27	0.32	0.26	0.22	0.45	

Notes:

#1: The gray boxes identify the lower and upper values for columns.

#2: Participants' RSR estimates differed

Table 3-6: Ultimate Strength Analysis Results: Direction 3 (315 degree from True North) -- with pile/soil considered

Participant	Base Shear: Section 17 Ult. Load S-17 (kips)	Base Shear: 20th Edition, Ref. level S-20 (kips)	Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
A (#2)	1,331	1,482	-	1,598	2,796	1.89	Pile Compression
B	2,209	1,900	847	1,197	1,861	0.98	First leg/ Pile yielding
C	2,295	1,860	-	2,100	2,430	1.31	Foundation
D	1,440	1,280	1,410	2,435	2,764	2.16	Foundation
E	2,360	2,246	-	-	-	-	-
F	2,162	1,680	-	1,905	2,545	1.51	Joint failure/ Jacket leg portal
G	1,907	2,325	-	1,060	1,550	0.67	Pile, Soil
H	2,449	1,982	1,274	1,554	2,895	1.46	Pile, deck legs, horz. braces
I	2,755	2,227	998	2,425	2,618	1.18	Jacket/Foundation
J	2,884	-	-	3,417	3,439	-	Legs and braces
K	2,702	2,268	-	1,605	2,039	0.90	Jacket
L	2,755	2,235	-	1,102	1,804	0.81	Jacket, piles
M	-	2,613	-	2,174	2,613	1.00	Soil
Mean	2,271	2,008	1,132	1,881	2,446	1.26	
St. Dev.	504	386	256	681	539	0.46	
COV	0.22	0.19	0.23	0.36	0.22	0.37	

Notes:
#1: The gray boxes identify the lower and upper values for columns.
#2: Participants' RSR estimates differed

Table 3-7: Voluntary Ultimate Strength Analysis Results -- Fixed Base Case

Participant	Direction-1 (225 degree from True North)					Direction-2 (270 degree from True North)					Direction-3 (315 degree from True North)				
	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism
B	-	-	-	-	-	2,070	3,441	4,683	2.26	Strut buckling	1,900	3,526	3,526	1.86	Brace buckling
C	1,660	3,490	3,870	2.33	Leg bending	2,230	3,505	4,870	2.18	Leg bending	1,860	3,465	3,800	2.04	Leg bending
D	-	-	-	-	-	1,310	1,100	2,446	1.87	Leg yielding	-	-	-	-	-
J	1,948	3,265	3,271	1.68	Legs and braces	3,050	2,591	2,853	0.94	Legs and braces	2,884	3,351	3,374	1.17	Legs and braces
K (#1, #2)	1,590	4,196	4,196	2.64	K-braces	2,420	4,057	4,740	1.96	Jacket legs	2,268	3,910	4,046	1.78	K-braces and joints
M	2,590	2,007	3,568	1.38	Leg yield/ K-braces	3,265	2,005	3,582	1.10	Leg yield, K-braces	2,613	2,211	3,512	1.34	Leg yield, K-braces
Mean	1,947	3,240	3,726	2.01		2,391	2,783	3,862	1.72		2,305	3,293	3,652	1.64	
St. Dev.	456	912	397	0.58		707	1,100	1,055	0.56		445	640	269	0.37	
COV	0.23	0.28	0.11	0.29		0.30	0.40	0.27	0.33		0.19	0.19	0.07	0.22	

Notes: #1: Assuming rigid joints

#2: Allowing for joint flexibility & P-Delta effects at joints for Direction 3, Ru = 4,077 kips and RSR = 1.8.

Table 3-8: Mean, Standard Deviation, and COV for Input Parameters and Analysis Results

Item	Section 17, Ultimate Strength Metocean Load Criteria			RP2A, 20th Edition, Metocean Load Criteria			Load at Ist Member with Linear IR = 1.0	Load at Ist Member with NonLinear Event	Ultimate Capacity,	Reserve Strength Ratio,
	Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Base Shear S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Base Shear, S-20 (kips)	S1 (kips)	(kips)	Ru (kips)	RSR = Ru/ S-20
Direction-1 (225 degree):										
Mean	58.31	2.68	2,001	55.68	2.57	1,735	1,052	1,831	2,513	1.51
St. Dev.	4.04	0.74	466	2.57	0.73	439	332	747	547	0.56
COV	0.07	0.28	0.23	0.05	0.28	0.25	0.32	0.41	0.22	0.37
Direction-2 (270 degree):										
Mean	65.77	3.69	2,699	62.29	3.44	2,210	1,086	1,616	2,209	1.07
St. Dev.	3.82	0.30	611	1.14	0.19	592	350	414	477	0.48
COV	0.06	0.08	0.23	0.02	0.06	0.27	0.32	0.26	0.22	0.45
Direction-3 (315 degree):										
Mean	63.92	3.41	2,271	60.22	3.22	2,008	1,132	1,881	2,446	1.26
St. Dev.	3.80	0.37	504	1.57	0.34	386	256	681	539	0.46
COV	0.06	0.11	0.22	0.03	0.11	0.19	0.23	0.36	0.22	0.37
Average COV for Three Directions	**0.06**	**0.16**	**0.23**	**0.03**	**0.15**	**0.24**	**0.29**	**0.34**	**0.22**	**0.40**

Section 4
Participants' Inquiries, Review and Feedback to the 92-5

4.1 INQUIRIES

Inquiries from participants, where significant, were provided in the form of Revisions to the Benchmark Basis Document. Revision 2 (dated April 12, 1994) and Revision 3 (dated April 20, 1994) included additional information based on response to these inquiries. More information on this is given in Section 2.1.

4.2 REVIEW AND FEEDBACK OF DRAFT SECTION 17 (PART B)

Few participants provided written comments to the Draft Section 17 document for use by the API TG. The API TG provided response to participants comments and queries, which is included in Appendix-B. The response by API TG 92-5 is reply to both Trial and Benchmark Participants comments on Section 17, thus also cover a number of other comments obtained from the Trial application participants. It is organized by Sections of Draft Section 17 document, to allow Benchmark participants to trace response to their comments.

The "correct" metocean criteria and force calculation procedure identified by the TG92-5 WG3 members, for analyzing the Benchmark platform, is also provided in the Appendix-B.

The comments received through Part B of their Benchmark Documents are provided below.

- Section 17.1 – General

  "A philosophical background for Section 17 should be added as introduction (subsection 17.1) explaining what we are trying to do, so that a user can appreciate why different wave heights (as compared to 100 year waves, 20th Edition) have to be used for design level or ultimate level checks as well as for different exposure categories."

- Section 17.6.2a – Gulf of Mexico Criteria

  Under Item 4b, in Figure 17.6.2-4, the caption should indicate that the directions and factors also apply to currents.

- Section 17.7 – Structural Analysis for Assessment

  In 17.7.2b and 17.7.3b it is recommended that the clauses read "software developed and *validated* for that purpose."

- Section 17.7.3 – Ultimate Strength Analysis Procedures

Guidelines to select suitable analysis method (linear global, local overload or global inelastic) given in Section 17.7.3a through 17.7.3c should be more clearly stated.

- Section 17.7.3c and C17.7.3c – Global Inelastic Analysis

Items 3.b and 3.c in Section 17.7.3c do not address the issue of modeling braces that carry significant moments. One example is braces that frame into pile heads.

Item 3.d in Section 17.7.3c does not clearly state what the actual loads or the loads based on the strength that act on joints. Some joint modeling techniques should be stated here with their advantages and disadvantages.

Section 17.7.3c provides instructions on element grouping and this is expanded significantly in the commentary. It is questioned whether the level of guidance in the guideline itself is helpful. It is suggested that the clause should reiterate the intention to use best estimate properties to model components (as stated explicitly for foundations) and indicate that, if required, further guidance on the grouping of similar element for modeling purposes is contained in the commentary.

The discussion regarding the modeling of structural members in the commentary appears to be written with the concepts of an "INTRA" type analysis in view. Other programs which have been developed and validated for ultimate strength analysis have automatic facilities to accommodate large deflection beam column action including the effects of end fixity without requiring the user to select specific K factors or element types before performing an analysis. It is also unnecessary to scrutinize working stress analysis results to establish which element types should be selected for each location "based on the dominant stresses." These software packages make the single step to ultimate strength check increasingly viable from economic and time standpoints.

Perhaps a more general approach would be to state that the modeling should properly account for beam column effects, the potential onset of plasticity, and the effect of frame restraints on buckling capacity, etc. This generality leaves the analyst better able to interpret the guideline and less likely to give inadequate consideration to factors which may cursorily be disregarded as irrelevant.

- Section C17.6.2 Wave/Current Deck Force Calculation Procedures

The presentation of deck loading could be open to different interpretation. For example wave loads on the net silhouette area are readily distributed equally to decks above and below. In reality structural members might share the load top to bottom whereas loads incident on equipment/structure standing on the deck will pass loads to the lower level almost exclusively. Should the net area modeling be associated with the net deck area for attracting loads rather than between deck silhouette. Alternatively, the proposed procedure may be adequate but should perhaps be flagged for further investigation in a sensitivity study should the margin beyond the required ultimate strength be small.

- Section C17.7.3c – Global Inelastic Analysis

  In Item 3.g, it is required that the gap between jacket and conductor be modeled. Clearly this is aimed at realism. However, there is uncertainty in the initial position of the conductor in the slot. For this reason the added complexity may not necessarily lead to an improved representation of the system behavior. Perhaps it need not routinely be modeled but if the criteria are only just met this and other factors such as initial member out-of-straightness etc. should be recommended for inclusion in a sensitivity study.

In addition, the following typographical errors were cited.

Section 17.3.1c	"platform *is* not"
Section 17.5.2	"environmental" - remove space and hyphen
Section 17.6.2b	"Section 17.6.2a.2" ? There is no Section 17.6.2a.5
Section 17.7.3	"to ensure adequacy"
Section 17.7.3c	"deformation"

4.3 OTHER COMMENTS AND OBSERVATIONS FROM PARTICIPANTS

Several participants commented on their results and discussed current limitations of modeling and analysis. Selected discussions from their documents are reproduced in this section.

- Joint Modeling

  One participant discussed the joint modeling issue as follows:

  > "The issue of joint modeling is not easily addressed by most nonlinear pushover analysis software and they do not have the capability to explicitly account for the joint can capacity in the ultimate strength analyses. In

previous analyses, we have addressed this issue by degrading the member capacities to match the joint can capacities. However, there are various uncertainties with this procedure. First, our experience is that the API joint can capacity formulation is generally conservative even after the safety factor is removed. Second, obviously as the joint cans fail, this will change the internal load distribution. So until the joint can capacity failure and load re-distribution algorithms are incorporated into the pushover analysis program, the simplified procedures for including the effect of joint can failures are at best first pass approximations. We therefore recommend further research in this area which would allow us to incorporate this capability into the ultimate strength analysis programs.

Another participant discussed the joint modeling issue as follows:

"Modeling joint behavior has been a difficult task. Results from past analyses have shown that some of the techniques used gave questionable results (Andrew JIP, Phase I). It has been proposed that joint modeling techniques should be studied carefully with some experimental backup.

- Wave/Current Loads on the Deck

One participant computed wave-in-deck loads for higher return periods (see Section 3.5.4) and commented as follows:

"In this analysis we have found that the ultimate strength for the orthogonal directions could vary significantly depending on how these loads are incremented from the 100-year loads to ultimate failure. In addition, these loads become an increasing component of the total base shear for the higher return periods. Therefore, further validation and calibration of the wave impact load algorithm are also important issues."

Section 5
Summary and Observations

Thirteen companies submitted their Benchmark Analyses. The benchmark platform is a 4-leg platform located in very soft to very stiff clay zone in 157 ft water depth, with 36 inch diameter piles penetrating 355 ft below the seabed. The platform was identified as a manned and with significant environmental impact upon its failure.

The participants selected metocean parameters for the appropriate category using API RP 2A WSD Section 17 and performed required ultimate capacity analysis. Several participants also submitted results for voluntary analyses suggested in the Benchmark Basis Document and for other analysis cases.

Nine different software packages were used by participants for nonlinear ultimate capacity analysis. These packages have been developed in the U.S.A., U.K., and Norway. They represent most of the packages available with the industry for performing nonlinear ultimate capacity pushover analysis of steel jacket offshore platforms. Not all of these software packages are completely integrated to perform a complete task from model generation to obtaining post-processing results. Thus, in some cases, other software or software external to the nonlinear analysis packages were used.

The results submitted were compared, and several descriptive tables and figures were developed to determine variations in the selected metocean parameters and the results obtained by the participants. A majority of participants performed analysis for three storm approach directions. Table 3-8 also provided the average variations (measured as the Coefficient of Variation) for the three directions for the key results of the benchmark. The results indicate significant variation among participants in the selected metocean parameters, the base shear values obtained for the Section 17 and 20th Edition criteria, and the ultimate capacity estimates. The component and platform failure modes obtained and the load levels when the first member experiences an IR of 1.0 or a nonlinear event occurs also varied significantly among participants. While a majority of participants found the platform failure mode to be pile yielding and inadequate soil capacity in compression, some participants found that soil and pile capacities were adequate and the jacket elements were weaker, thereby governing ultimate capacity and the platform failure mode.

Several key observations regarding the results of the benchmark are as follows:

Hydrodynamic Loadings

The average variation in hydrodynamic loads (base shear) was about 24 percent. This variation is higher than was initially expected. This is in part due to use of the RP 2A 20th Edition hydrodynamic loading procedures which have only just begun to be used by most organizations. The 20th Edition procedures consider directional variation of wave height and current, doppler shift in wave period, current blockage, conductor shielding, variable drag and inertia coefficients, etc. which were not contained in prior RP 2A procedures. These features resulted in much of the increased variation of results.

The average variation in selecting the specific wave height and current was much smaller (6 and 16 percent); however, considering that these values can supposedly be selected directly from figures and tables in RP 2A, it is surprising that there is any variation at all.

Close study of the results indicates that even for those participants selecting the same wave height there is still variation in base shear, although not as much as with all participants. The variation in this case is likely due to differences in the wave force computation procedures, selection of drag and inertia coefficients, wave theory (kinematics) and three dimensional computer modeling of the platform.

The highest variability of any parameter studied was wave loads on the deck. For the Section 17 ultimate strength wave case 9 of 13 participants provided wave loads on the deck. The participants' wave-in-deck load estimates ranged from 2 to 18 percent of the total Section 17 base shear for the three directions. The average variability (COV) among the reported wave-in-deck loads exceeded 100 percent. A contributing factor is the sensitivity of the procedure provided in Section 17 to wave crest elevation. Small changes in wave crest elevation (e.g. 1 ft) result in large changes in loads. Since the wave crest elevation is based upon multiple parameters such as wave theory, wave period, storm tide, etc., there is bound to be variation among participants.

The variation in hydrodynamic loads may reduce with time as the RP 2A, 20th Edition procedure is used repeatedly and is more thoroughly understood by organizations. Changes should also be considered for the RP 2A Section 2 description and Section 17 description of the hydrodynamic procedures (including wave loads on decks) so that they are more clear, are easier to understand and result in more consistent results between different organizations.

Appendix B includes the applicable metocean criteria and wave force procedures to the Benchmark platform analysis developed by the API TG WG3 after the October 19 project meeting. This would identify various reasons for variations among participants.

Platform Capacity

The average variation in ultimate capacity was 22 percent (soils included). The results tended to scatter close to a central value, with several "outlier" values a good distance from the mean. For example, for direction 2 (diagonal) Figure 3-20 shows several outlier values in terms of participant J, G and B. Eliminating these three participants reduces the variation to 12 percent.

The average variation in RSR is 40 percent. Since RSR is determined from the base shear and ultimate capacity, it includes the variation in each of these values (24 and 22 percent respectively), resulting in a high total variation.

Similar to computing hydrodynamic loads, the above variations in ultimate capacity will likely reduce with time as more organizations become familiar with the process. As noted below, several participants commented that more direction would be helpful in Section 17 related to ultimate capacity procedures. Such direction would also tend to reduce the variation.

Pass/Fail Assessment of Platform

A comparison of ultimate capacity with the Section 17 base shear in Tables 3-4 to 3-6 indicates significant variation in a pass/fail assessment of this platform by the various participants. In Direction 1, in which results from 7 participants are available, 5 participants indicate the platform will pass and 2 participants indicate it will fail per Section 17 ultimate strength requirements (when the platform is under Significant Environmental Impact category). For Direction 2, only 2 participants indicate it will "pass" and 9 participants indicate it will "fail." Similarly, for Direction 3, only 6 participants will classify it under "pass" category.

Participant Feedback to API TG 92-5

Participant feed back was focused primarily around the procedures (or lack of) contained in Section 17 related to ultimate strength analysis. Feedback on the general approach in Section 17 can be found in the Trial Applications Final report. Specific comments by participants addressed the philosophy of Section 17, lack of clear procedures for nonlinear platform modeling, wave-in-deck force calculation procedures, joint capacity and joint modeling.

There were surprisingly no comments on foundation modeling, but this is perhaps due to the fact that RP 2A currently provides a procedure to develop nonlinear soil spring characteristics that can be used for ultimate strength analysis. There were also few comments on the 20th edition wave load recipe, when in fact a review of results indicates a high variation in hydrodynamic base shear between participants, which is most likely due in part to incorrect interpretation of the RP 2A procedure.

A majority of the comments request that additional information be contained in Section 17 and related commentary that address ultimate strength analysis procedures. Joints in particular were singled out as an area where further information and guidance would be helpful. However, additional information may be difficult at this time for TG 92-5 to accommodate in Section 17. Ultimate strength analysis is still an ever-changing methodology, with different organizations using different approaches to solve the same problem. There is no one "accepted" set of procedures or techniques for determining platform capacity. In addition, a majority of the previous work in this technical area was developed under confidential studies or is contained within proprietary software. Until the industry as a whole reaches some consensus on an "accepted" approach for ultimate strength evaluation it seems that it will be difficult for API to provide further guidance within Section 17.

API TG 92-5 Response to the Participants' Comments:

The response received from the API TG 92-5 (Appendix-B) clarifies the various issues raised by the participants. The "correct" metocean criteria and force calculation procedure (Appendix-B), identified by the TG92-5 WG3 members for evaluating the Benchmark platform also clarify some of these issues.

Comparison of the Original and Revised Results:

The re-submittals from participants (Appendix E) and information identifying reasons for variations and errors (Appendix C) are summarized in Appendix A. The Tables 3-1 to 3-8 and Figures 3-17 to 3-19 were revised for effect of new information. Table A-8 and Figures A-17 to A-19 provide a comparison of the original and revised values for key quantities. Most of the revisions were made by the participants to their metocean parameters and load estimates. Only one participant revised their capacity estimates and a few revised the failure modes.

These results indicate that the average variation decreases for wave height (3 percent) but remains same for the current (15 percent). A comparison of participants values (Tables A-1 to A-3) with the API TG selected values (Appendix B), and participants' response (Appendix-C) indicate that the values per a number of participants differ from "correct" values due a number of reasons. To a large extent the differences appear to relate to understanding of the Section 17 and RP 2A, 20th Edition procedures.

The average variation in the base shear decreases significantly (12 percent). Figures A-1 to A-3 indicate that the range of base shear among participants for each direction is very significant.

The average variations and ranges in the ultimate capacity (23 percent), load levels at first member with linear IR of 1.0 and at a nonlinear event do not change from the original values. Due to reduction in variation in the 20th Edition base shear the average variation in RSR also reduces to 23 percent.

These results are encouraging and indicate that further coordinated effort by the API TG would be able to identify in more detail the reasons for variations in metocean parameters, loads and capacity estimates. Such information would be useful for the API TG for decisions on revisions to the API RP 2A and Section 17 documents, if necessary.

Appendix A
Modified Analysis Results

Following the Final Meeting on October 19 four participants (A, B, D, K) provided resubmittal documents. Some participants explicitly identified "gross errors" in their results whereas several others the likely reasons for variations in their results from those of other participants. The effect of this information to Figures 3-9, 3-17 to 3-20 and changes from the original submittals are presented in this Appendix. The variations have been briefly discussed in Section 5.

Copies of participants' response and identification of the reasons for variations in their results are provided in Appendix C. The following provides summary of the reasons as identified by the participants (see Appendix C for details):

Participant A:

- Errors and misinterpretations
- "Gross errors" made in input into the analysis model
- Oversight and relative difficulty in interpreting Section 17 and 20th metocean criteria resulted in low wave heights.
- Pile/soil axial "t-z" data incorrectly input resulting in high initial stiffness in the load-displacement curves.
- Error in development of "p-y" curves per the API RP 2A procedures.
- Error in statement (in original submittal) that piles were assumed grouted in legs.
- Conductors were modeled to contribute to foundation capacity.

Participant B:

- Direction-1 and Direction-3 results got switched in the original submittal.
- Incorrect longitude used resulted in different current speed and base shear.
- Change in failure mode for Direction-2.

Participant D:

- Engineers' first use of software and API RP 2A, 20th edition methodology.
- Difference in wave profile generation approach for use in the pushover analysis.

Participant F:

- Considered directionality effect but based on engineering judgment decided to use same wave height for Directions 2 and 3. The decision was based on high degree of uncertainties in extreme wave approach direction and in the survey of platform orientation.

Participant G:

- Wave-in-deck forces considered only when the wave crest exceeded Elev. (+) 42.13'. Other participants may have considered wave-in-deck forces from Elev. (+) 33'.
- Modeled conductors as wave load elements, which may have resulted in lower ultimate capacity. Agree that modeling the conductors as foundation elements is an acceptable practice, particularly for the 4 leg (36" pile) benchmark platform.
- Used cyclic "p-y" curves to define the soil lateral capacity, since it was considered that cyclic criteria is more commonly used by other operators and design consultants. Participant G advocates use of static "p-y" curve formulations for ultimate capacity analysis. It is likely that some other JIP participants did use static "p-y" curves, contributing to higher calculated ultimate capacity.
- If participant G included the well conductors in the foundation model and had used static "p-y" curves for the soil lateral capacity, a much higher ultimate capacity would have been achieved.

Participant I:

- Interpolated wave height factors between two principal wave directions.
- Neglected wave load on deck for simplicity.
- Used simplified modeling for the conductor framing.
- Considered conductors supported at the mudline and modeled as hinged at mudline. This resulted in horizontal diagonals becoming first members with I.R. of 1.0 and a portion of the load was transmitted to the hinge support through that member.

Participant J:

- Software used did not provide wave load on jacket and wave-in-deck loads separately.
- No wave-in-deck loading was calculated due to restrictions in the software.
- Conductors were modeled not to carry horizontal loads and also not to contribute to the foundation capacity.
- The software used computes the utility ratio based on ultimate strength of the member and not based on allowable stresses specified in codes with safety factors included. Thus the load level at first member with I.R. is the same as the load level of first member failure.
- For the fixed case, the model was taken to be fully fixed in all directions at the mudline level.

Participant K:

- Used linear interpolation of the values in RP 2A, 20th Edition rather than the prescribed +/- 22.5 degrees.
- Wind load based on the 20th Edition instead of Section 17.
- Used the centroild of wind area slightly offset.
- Wave blockage factor was assumed to be 0.845 for all directions.
- Modeled conductor grid with two "equivalent" members attached to be major horizontal framing, which may have resulted in early failure of horizontal members of the lowest framing.
- Modeled conductors pinned laterally and released vertically at the guided for hydrodynamic loading and stiffness. Below the mudline modeled them with static "p-y" curves.
- For the fixed case, fixed (all six degrees of freedoms) jacket legs and piles at 12 ft. below the mudline.

Participant M:

- Base shear did not account for the wave kinematic and current blockage factors. Provided revised values.

	Range	Mean	Cov (%)	Per API TG
Wave Height Section 17 (Ult. Str.)	50.90 55.8 61.2#1 64.60#2	59.47	4 (7)#3	61.2
Wave Height RP2A, 20th Ed.	51.72 56.7 59.85	55.78	3 (5)	56.7

a) Metocean Criteria - Wave Height (ft)

	Range	Mean	Cov (%)	Per API TG
In-Line Current Speed Section 17 (Ult. Str.)	1.7 3.88	2.83	28 (27)	2.31
In-Line Current Speed RP2A, 20th Ed.	1.56 3.54	2.49	25 (28)	2.11

b) Metocean Criteria - Current Speed (ft/sec)

	Range	Mean	Cov (%)
Base Shear Section 17 (Ult. Str.)	1,243 1,880 2,780	2,150	14 (23)
Base Shear RP2A, 20th Ed. (S_{ref})	1,056 1431 2174 2,590	1,764	14 (25)
Load @ First Member with NLinear Event	1,119 1,197 3,527	1,921	39 (41)
Ultimate Capacity (R_u)	1,610 3,573	2,487	24 (22)

c) Analysis Results - Load Levels (kips)

	Range	Mean	Cov (%)
Reserve Strength Ratio (RSR)	0.74 1.69 2.47	1.32	24 (37)

d) Analysis Results - RSR

#1 : Revised Ranges

#2 : Original Submittal Ranges (See Section 3)

#3 : Original Submittal COV's

Figure A-1 Summary of Variations of Metocean Parameters and Analysis Results - Direction-1

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	Range	Mean	Cov (%)	Per API TG
Wave Height Section 17 (Ult. Str.)	54.80#2 64.02#1 68.00	67.15	2 (2)#3	68
Wave Height RP2A, 20th Ed.	60.20 63.00	62.68	1 (2)	63

a) **Metocean Criteria - Wave Height (ft)**

	Range	Mean	Cov (%)	Per API TG
In-Line Current Speed Section 17 (Ult. Str.)	3.00 3.31 3.88	3.80	4 (8)	3.83
In-Line Current Speed RP2A, 20th Ed.	3.02 3.54	3.47	4 (6)	3.49

b) **Metocean Criteria - Current Speed (ft/sec)**

	Range	Mean	Cov (%)
Base Shear Section 17 (Ult. Str.)	1,495 2,234 3,429	2,921	11 (23)
Base Shear RP2A, 20th Ed. (S_{ref})	1,150 1713 2810 3,265	2,316	12 (27)
Load @ First Member with NLinear Event	980 2,295	1,639	23 (26)
Ultimate Capacity (R_u)	1,496 3,143	2,107	23 (22)

c) **Analysis Results - Load Levels (kips)**

	Range	Mean	Cov (%)
Reserve Strength Ratio (RSR)	0.57 1.13 2.21	0.88	20 (45)

d) **Analysis Results - RSR**

#1 - Revised Ranges

#2 - Original Submittal Ranges (See Section 3)

#3 - Original Submittal COV's

Figure A-2 Summary of Variations of Metocean Parameters and Analysis Results - Direction-2

:046\TRIALS\FIG-A-2

	Range	Mean	Cov(%)	Per API TG
Wave Height Section 17 (Ult. Str.)	53.10[#2] 63.7[#1] 68.00	65.09	2 (6)[#3]	64.6
Wave Height RP2A, 20th Ed.	56.70 59.85 63.00	60.42	2 (3)	59.9

a) Metocean Criteria - Wave Height (ft)

	Range	Mean	Cov(%)	Per API TG
In-Line Current Speed Section 17 (Ult. Str.)	2.88 3.00 3.88	3.41	12 (11)	3.21
In-Line Current Speed RP2A, 20th Ed.	2.63 2.80 3.54	3.11	12 (11)	2.94

b) Metocean Criteria - Current Speed (ft/sec)

	Range	Mean	Cov(%)
Base Shear Section 17 (Ult. Str.)	1,331 1907 2,884	2,441	12 (22)
Base Shear RP2A, 20th Ed. (S_{ref})	1,280 1680 2325 2,613	2,034	11 (19)
Load @ First Member with NLinear Event	1,060 3,417	1,866	37 (36)
Ultimate Capacity (R_u)	1,550 3,439	2,399	22 (22)

c) Analysis Results - Load Levels (kips)

	Range	Mean	Cov(%)
Reserve Strength Ratio (RSR)	0.67 1.51 2.16	1.16	24 (37)

d) Analysis Results - RSR

#1 - Revised Ranges

#2 - Original Submittal Ranges (See Section 3)

#3 - Original Submital COV's.

Figure A-3 Summary of Variations of Metocean Parameters and Analysis Results - Direction-3

.046\TRIALS\FIG-A-3

Component Failure Modes	Participant												
	A	B	C	D	E	F	G	H	I	J	K	L	M
Jacket Components													
Leg Yielding		Yes	Yes	Yes	Yes			Yes		Yes			
K-brace										Yes			
K-joint					Yes								
Horizontals					Yes			Yes	Yes		Yes	Yes	
Pile Sections:													
First Yielding	Yes	Yes						Yes				Yes	
Full Yielding		Yes											
Double Hinging	Yes					Yes	Yes						
Soil Capacity:													
Compression Capacity	Yes	Yes	Yes	Yes	Yes		Yes		Yes		Yes	Yes	Yes
Tensile Capacity													
Platform Failure Mode (#1)	Soil Capacity	Pile Yielding/ Soil Bearing	Pile Plunging	Foundation (Soil Capacity)	Soil Capacity	Foundation/ Pile double hinging	Pile/ Soil	Pile Yielding	Jacket/ Foundation	Legs & Braces	Foundation T-Z	Jacket-Pile	Soil

Note #1: Per Participants' Original or Revised Submittals

Figure A-4: Comparison of Component and Platform Failure Modes - Direction 2 (270 degree from True North)

Wave Height (ft.)		< 1,500 (V. Low)	1,501-2,000 (Low)	2,001-2,500 (Medium)	2,501-3,000 (High)	> 3,000 (V. High)
V. High	> 65					
High	63.1 - 65					
Medium	63		F	A,B,C,D,E,H,L	G,M	
Low	60 - 62.9			I, K		
V. Low	< 60					

20th Edition Base Shear (Kips)

Participant J did not provide sufficient information (Ref. Table A-2)
to be included in this chart

a) Based on Selected Wave Height (20th Edition) and Base Shear

Ultimate Capacity (Kips)		< 1,500 (V. Low)	1,501-2,000 (Low)	2,001-2,500 (Medium)	2,501-3,000 (High)	> 3,000 (V. High)
V. High	> 2,700					
High	2,301-2,700			E, H, I	M	
Medium	1,901-2,300		F	C,D		
Low	1,501-1,900			A,K,L	G	
V. Low	< 1,500			B		

20th Edition Base Shear (Kips)

Participant J did not provide sufficient information (Ref. Table A-5) and
to be included in this chart

b) Based on Reference Level Base Shear and Ultimate Capacity

Figure A-5: Classification Based on Wave Height and Analysis Results-Direction 2

Table A-1: Comparison of Metocean Parameters and Loads: Direction 1 (225 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	In-Line Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	In-Line Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	225	60.70	3.88	1,943	156	2,099	56.70	2.08	1,386	45	1,431
B	225	61.00	2.60	1,802	118	1,920	56.70	2.38	1,514	106	1,620
C	225	61.20	2.40	1,895	100	1,995	56.70	2.19	1,570	90	1,660
D	225	61.20	2.50	2,155	91	2,246	56.70	2.28	1,803	47	1,850
E	225	61.20	1.70	2,732	48	2,780	56.70	1.56	2,028	23	2,051
F	-	-	-	-	-	-	-	-	-	-	-
G (#3)	225	60.30	3.88	2,604	0	2,604	56.70	3.54	2,174	0	2,174
H (#2)	225	61.20	3.11	1,860	95	2,038	56.70	2.84	1,535	60	1,669
I (#2)	225	56.23	2.44	1,940	0	1,992	52.48	2.23	1,600	0	1,600
J	225	55.80	2.06	1,948	0	1,948	-	-	-	-	-
K (#4)	225	55.83	3.76	1,840	40	1,880	51.72	2.32	1,380	210	1,590
L	-	-	-	-	-	-	-	-	-	-	-
M	245	-	-	-	-	-	56.70	3.50	1,928	65	1,993
Mean		59.47	2.83	2,072	65	2,150	55.78	2.49	1,692	65	1,764
St. Dev.		2.44	0.78	330	55	306	1.95	0.63	276	62	240
COV		0.04	0.28	0.16	0.85	0.14	0.03	0.25	0.16	0.96	0.14

Notes:

#1: The gray boxes identify the participants revising their original submittal results

#2: Total base shear includes wind load on the deck

#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.

#4: Considered wave-in-deck loads above El. (+) 16'.

Table A-2: Comparison of Metocean Parameters and Loads: Direction 2 (270 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	270	67.45	3.88	2,105	390	2,495	63.00	3.49	1,791	236	2,027
B	270	68.00	3.86	2,391	486	2,877	63.00	3.53	2,002	118	2,120
C	270	68.00	3.86	2,580	355	2,935	63.00	3.52	2,065	165	2,230
D	270	68.00	3.87	2,658	450	3,108	63.00	3.54	2,269	133	2,402
E	270	68.00	3.67	3,152	78	3,230	63.00	3.37	2,380	71	2,451
F (#2)	270	68.00	3.88	1,783	364	2,234	63.00	3.54	1,481	155	1,713
G (#3)	270	67.00	3.88	3,330	99	3,429	63.00	3.54	2,810	0	2,810
H (#2)	270	68.00	3.31	2,568	340	2,992	63.00	3.02	2,134	110	2,318
I (#2)	270	64.53	3.86	2,872	0	2,951	60.23	3.52	2,367	0	2,367
J	270	64.02	3.80	3,050	0	3,050	-	-	-	-	-
K (#4)	270	66.84	3.87	2,632	198	2,830	61.93	3.52	1,994	426	2,420
L	270	68.00	3.88	2,782	140	2,922	63.00	3.54	2,296	123	2,419
M	290	-	-	-	-	-	63.00	3.54	2,391	128	2,519
Mean		67.15	3.80	2,659	242	2,921	62.68	3.47	2,165	139	2,316
St. Dev.		1.41	0.17	434	175	313	0.83	0.15	338	112	274
COV		0.02	0.04	0.16	0.73	0.11	0.01	0.04	0.16	0.81	0.12

Notes:
#1: The gray boxes identify the participants revising their original submittal results
#2: Total base shear includes wind load on the deck
#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.
#4: Considered wave-in-deck loads above El. (+) 16'.

Table A-3: Comparison of Metocean Parameters and Loads: Direction 3 (315 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	315	64.08	3.88	2,060	218	2,278	59.85	2.92	1,641	126	1,767
B	315	64.60	2.88	1,981	335	2,316	59.85	2.63	1,665	131	1,796
C	315	64.60	3.06	2,150	145	2,295	59.90	2.80	1,740	120	1,860
D	315	64.60	3.00	2,287	142	2,429	59.90	3.54	1,914	80	1,994
E	315	64.60	3.39	2,313	47	2,360	59.90	3.14	2,202	44	2,246
F (#2)	315	68.00	3.88	1,788	309	2,162	63.00	3.54	1,492	130	1,680
G (#3)	315	63.70	3.88	1,907	0	1,907	59.90	3.54	2,325	0	2,325
H (#2)	315	64.60	3.11	2,196	165	2,449	59.85	2.84	1,815	85	1,982
I (#2)	315	65.61	3.02	2,697	0	2,755	61.24	2.75	2,227	0	2,227
J	315	65.12	3.30	2,884	0	2,884	-	-	-	-	-
K (#4)	315	66.91	3.82	2,452	250	2,702	61.99	2.77	1,859	409	2,268
L	315	64.60	3.69	2,628	127	2,755	59.85	3.37	2,123	112	2,235
M	335	-	-	-	-	-	59.85	3.47	1,920	108	2,028
Mean		65.09	3.41	2,279	145	2,441	60.42	3.11	1,910	112	2,034
St. Dev.		1.22	0.40	335	118	285	1.07	0.36	261	105	224
COV		0.02	0.12	0.15	0.81	0.12	0.02	0.12	0.14	0.93	0.11

Notes:
#1: The gray boxes identify the participants revising their original submittal results
#2: Total base shear includes wind load on the deck
#3: Considered wave-in-deck loads above cellar deck B.O.S. Elev. (+) 42.13'.
#4: Considered wave-in-deck loads above El. (+) 16'.

Table A-4: Ultimate Strength Analysis Results: Direction 1 (225 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A	2,099	1,431	-	-	-	-	-
B	1,920	1,620	847	1,197	1,861	1.15	First leg/ Pile yielding/Soil bearing
C	1,995	1,660	-	1,990	2,350	1.42	Foundation
D	2,246	1,850	1,375	2,294	2,623	1.42	Foundation
E	2,780	2,051	-	-	-	-	-
F	-	-	-	-	-	-	-
G	2,604	2,174	-	1,317	1,610	0.74	Pile, Soil
H	2,038	1,669	1,260	1,630	2,827	1.69	Pile/horz. braces/deck leg
I	1,992	1,600	930	1,494	2,490	1.56	Jacket/Foundation
J	1,948	-	-	3,527	3,573	-	Legs and braces
K	1,880	1,590	-	-	-	-	-
L	-	-	-	-	-	-	-
M	-	1,993	-	1,921	2,564	1.29	Soil
Mean	2,150	1,764	1,103	1,921	2,487	1.32	
St. Dev.	306	240	254	744	598	0.31	
COV	0.14	0.14	0.23	0.39	0.24	0.24	

Notes: #1: The gray boxes identify the participants revising their original submittal results

Table A-5: Ultimate Strength Analysis Results: Direction 2 (270 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A	2,495	2,027	-	1,560	1,747	0.86	Soil capacity
B	2,877	2,120	807	1,166	1,496	0.71	First leg/ Pile yielding/Soil bearing
C	2,935	2,230	-	1,920	2,070	0.93	Foundation
D	3,108	2,402	1,500	1,990	2,200	0.92	Foundation
E	3,230	2,451	-	1,290	2,381	0.97	Soil capacity
F	2,234	1,713	-	1,937	1,937	1.13	Foundation/ pile double hinging
G	3,429	2,810	-	980	1,610	0.57	Pile, Soil
H	2,992	2,318	1,252	1,636	2,628	1.13	Pile, leg, horiz. braces
I	2,951	2,367	785	1,770	2,361	1.00	Jacket/ Foundation
J	3,050	-	-	2,295	3,143	-	Legs and braces
K	2,830	2,420	-	1,549	1,689	0.70	Foundation t-z
L	2,922	2,419	-	1,315	1,689	0.70	Jacket, Pile
M	-	2,519	-	1,900	2,446	0.97	Soil
Mean	2,921	2,316	1,086	1,639	2,107	0.88	
St. Dev.	313	274	350	377	478	0.18	
COV	0.11	0.12	0.32	0.23	0.23	0.20	

Notes: #1: The gray boxes identify the participants revising their original submittal results

Table A-6: Ultimate Strength Analysis Results: Direction 3 (315 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at Ist Member with Linear IR = 1.0 S1 (optional)	Load at Ist Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A	2,278	1,767	-	1,430	2,120	1.19	Pile Compression
B	2,316	1,796	641	1,186	1,964	1.09	First leg/ Pile yielding/Soil bearing
C	2,295	1,860	-	2,100	2,430	1.31	Foundation
D	2,429	1,994	1,410	2,435	2,764	1.39	Foundation
E	2,360	2,246	-	-	-	-	-
F	2,162	1,680	-	1,905	2,545	1.51	Joint failure/ Jacket leg portal
G	1,907	2,325	-	1,060	1,550	0.67	Pile, Soil
H	2,449	1,982	1,274	1,554	2,895	1.46	Pile, deck legs, horz. braces
I	2,755	2,227	998	2,425	2,618	1.18	Jacket/ Foundation
J	2,884	-	-	3,417	3,439	-	Legs and braces
K	2,702	2,268	-	1,605	2,043	0.90	Foundation t-z
L	2,755	2,235	-	1,102	1,804	0.81	Jacket, piles
M	-	2,028	-	2,174	2,613	1.29	Soil
Mean	2,441	2,034	1,081	1,866	2,399	1.16	
St. Dev.	285	224	340	690	526	0.27	
COV	0.12	0.11	0.31	0.37	0.22	0.24	

Notes: #1: The gray boxes identify the participants revising their original submittal results

Table A-7: Voluntary Ultimate Strength Analysis Results -- Fixed Base Case

Participant	Direction-1 (225 degree from True North)					Direction-2 (270 degree from True North)					Direction-3 (315 degree from True North)				
	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at Ist Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism
B (#1)	1,620	3,526	3,526	2.18	Brace buckling	2,120	3,441	4,683	2.21	Strut buckling	-	-	-	-	-
C	1,660	3,490	3,870	2.33	Leg bending	2,230	3,505	4,870	2.18	Leg bending	1,860	3,465	3,800	2.04	Leg bending
D (#1)	-	-	-	-	-	2,402	1,100	3,816	1.59	Leg yielding	-	-	-	-	-
J (#1)	-	3,265	3,271	-	Legs and braces	-	2,591	2,853	-	Legs and braces	-	3,351	3,374	-	Legs and braces
K (#2, #3)	1,590	4,196	4,196	2.64	K-braces	2,420	4,057	4,740	1.96	Jacket legs	2,268	3,910	4,046	1.78	K-braces and joints
M (#1)	1,993	2,007	3,568	1.79	Leg yield/ K-braces	2,519	2,005	3,582	1.42	Leg yield, K-braces	2,028	2,211	3,512	1.73	Leg yield, K-braces
Mean	1,716	3,240	3,726	2.25		2,338	2,783	4,091	1.87		2,052	3,234	3,683	1.85	
St. Dev.	187	800	356	0.35		160	1,100	806	0.35		205	724	300	0.17	
COV	0.11	0.25	0.10	0.16		0.07	0.40	0.20	0.19		0.10	0.22	0.08	0.09	

Notes: #1: The gray boxes identify the participants revising their original submittal results

#2: Assuming rigid joints

#3: Allowing for joint flexibility & P-Delta effects at joints for Direction 3, Ru = 4,077 kips and RSR = 1.8.

Table A-8: Mean, Standard Deviation, and COV for Input Parameters and Analysis Results

Item	Section 17, Ultimate Strength Metocean Load Criteria			RP2A, 20th Edition, Metocean Load Criteria			Load at Ist Member with Linear IR = 1.0 S1 (kips)	Load at Ist Member with NonLinear Event (kips)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20
	Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Base Shear S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Base Shear, S-20 (kips)				
Direction-1 (225 degree):										
Mean	59.47	2.83	2,150	55.78	2.49	1,764	1,103	1,921	2,487	1.32
St. Dev.	2.44	0.78	306	1.95	0.63	240	254	744	598	0.31
COV	0.04	0.28	0.14	0.03	0.25	0.14	0.23	0.39	0.24	0.24
Direction-2 (270 degree):										
Mean	67.15	3.80	2,921	62.68	3.47	2,316	1,086	1,639	2,107	0.88
St. Dev.	1.41	0.17	313	0.83	0.15	274	350	377	478	0.18
COV	0.02	0.04	0.11	0.01	0.04	0.12	0.32	0.23	0.23	0.20
Direction-3 (315 degree):										
Mean	65.09	3.41	2,441	60.42	3.11	2,034	1,081	1,866	2,399	1.16
St. Dev.	1.22	0.40	285	1.07	0.36	224	340	690	526	0.27
COV	0.02	0.12	0.12	0.02	0.12	0.11	0.31	0.37	0.22	0.24
Average COV for Three Directions (Revised)	**0.03**	**0.15**	**0.12**	**0.02**	**0.14**	**0.12**	**0.29**	**0.33**	**0.23**	**0.23**
Average COV for Three Directions (Original)	**0.06**	**0.16**	**0.23**	**0.03**	**0.15**	**0.24**	**0.29**	**0.34**	**0.22**	**0.40**

PARTICIPANT A

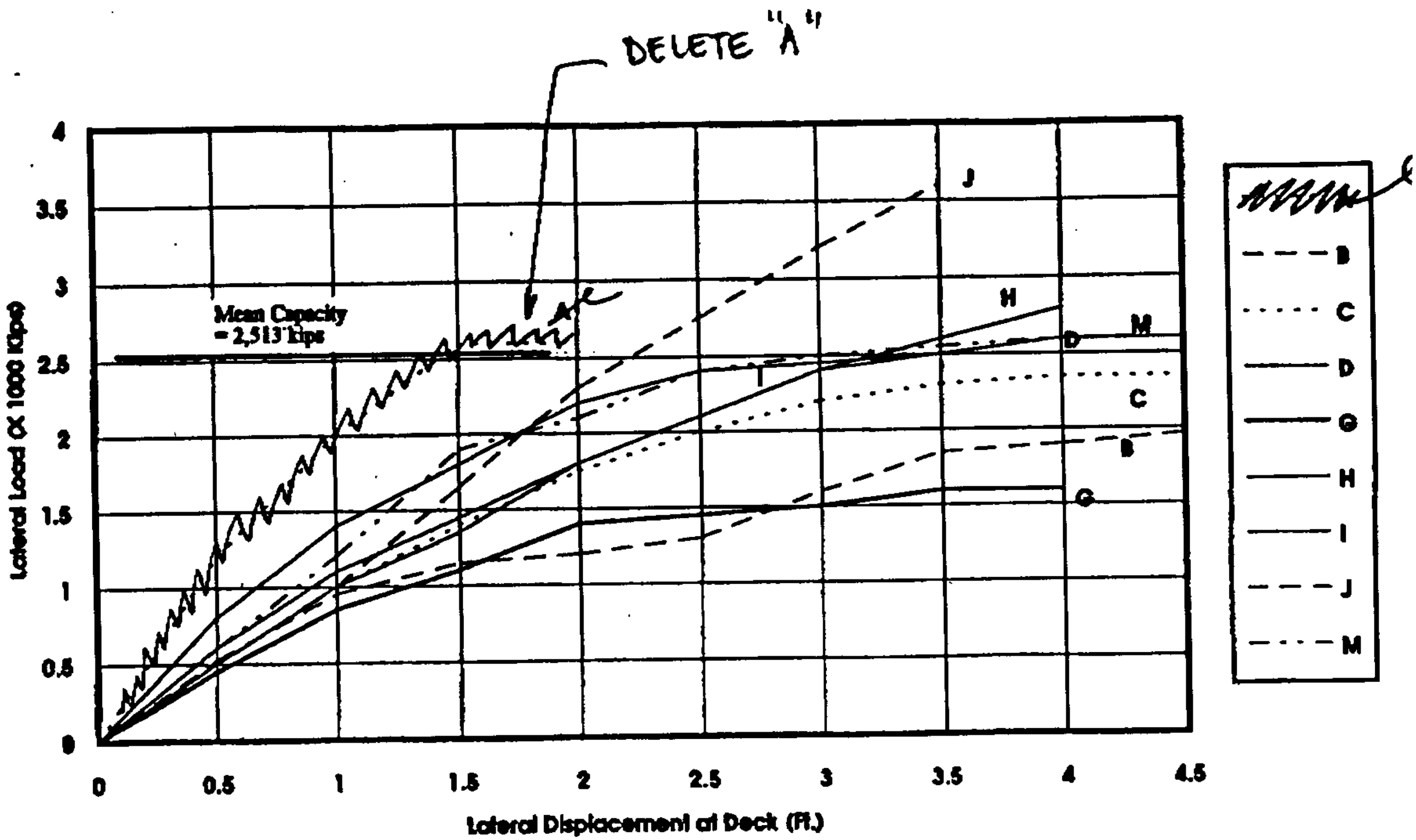

Figure 3-10 Load-Displacement Behavior - Direction 1

PARTICIPANT A

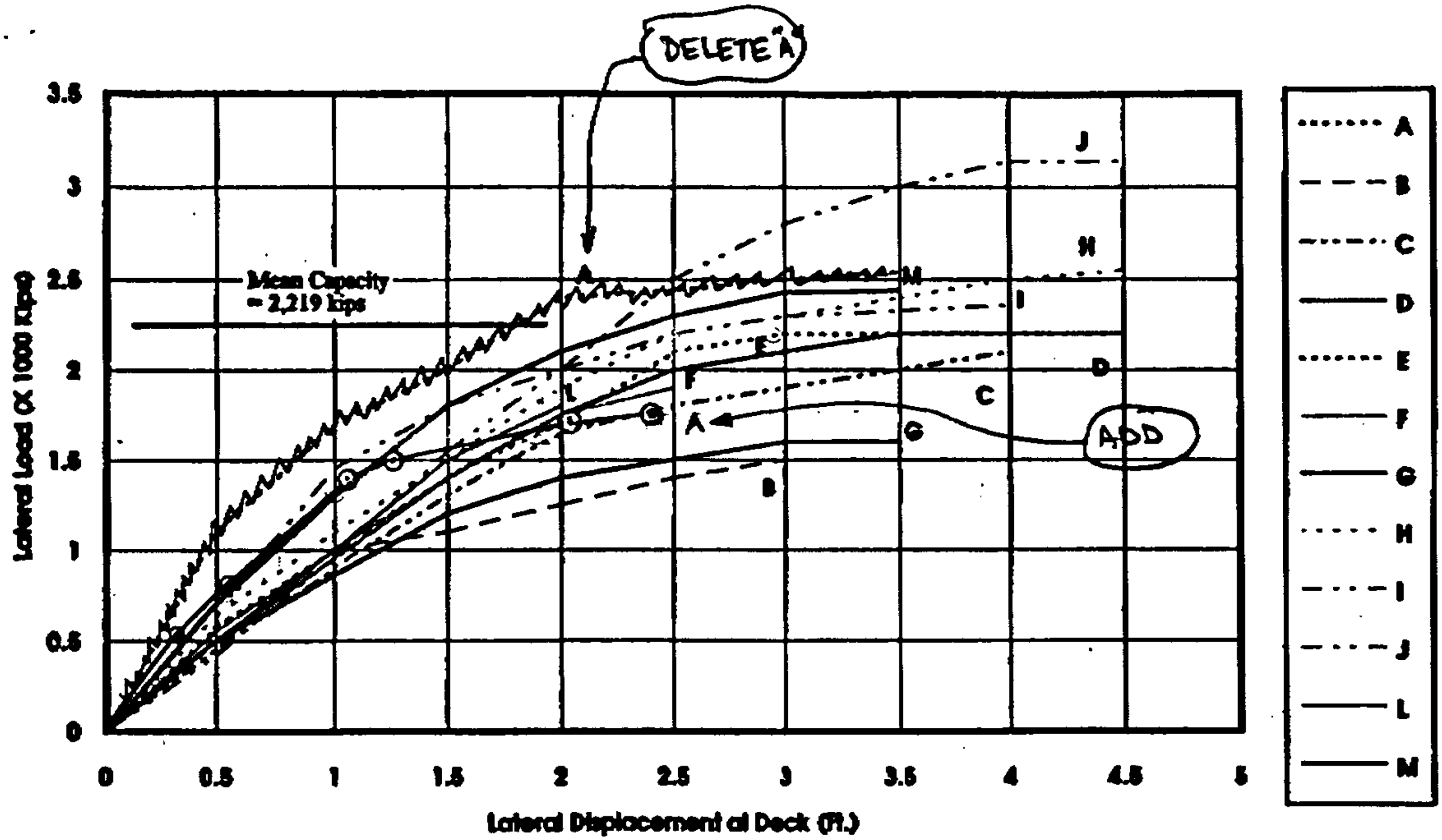

Figure 3-11 Load-Displacement Behavior - Direction 2

PARTICIPANT A

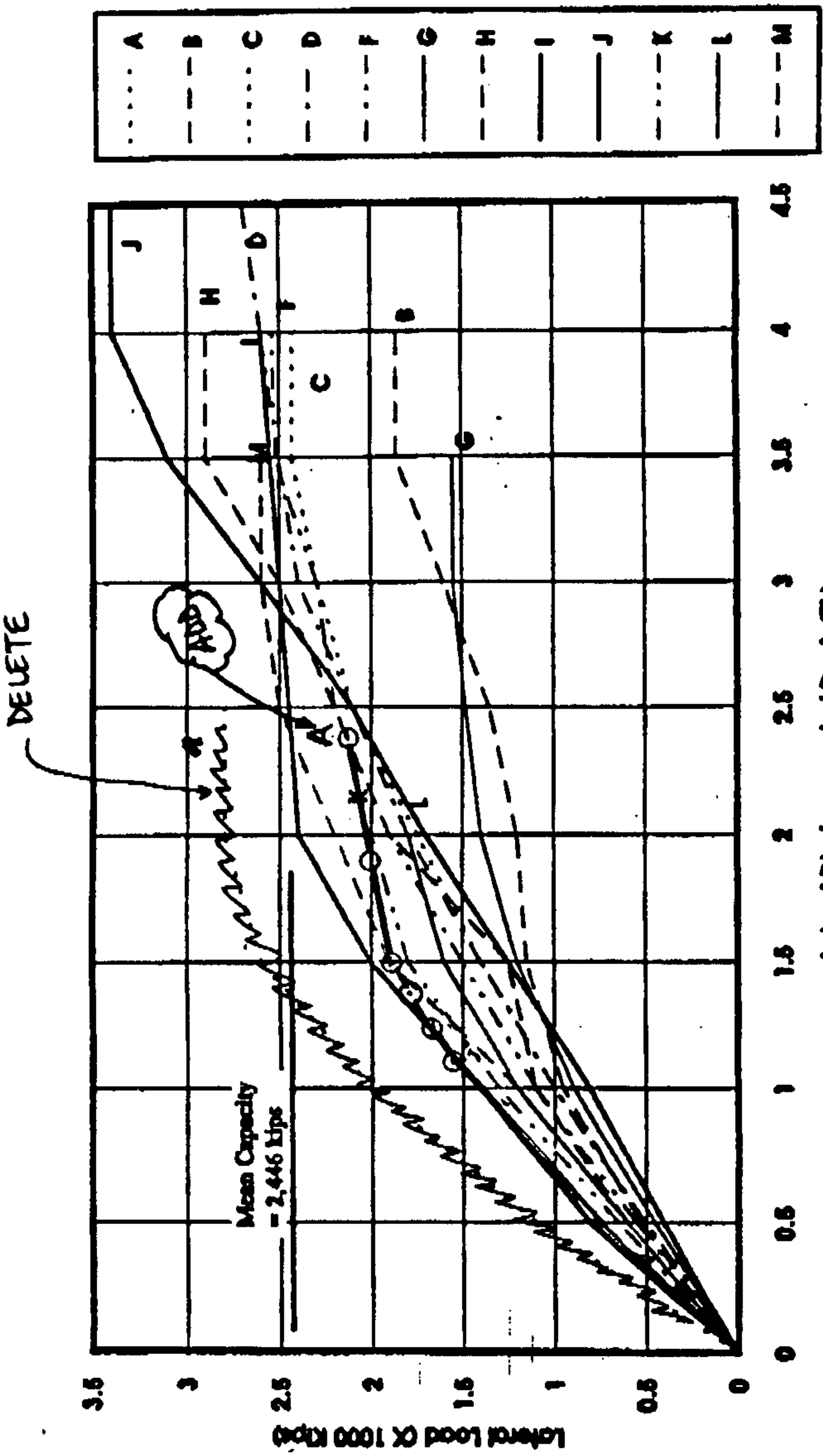

Figure 3-12 Load-Displacement Behavior - Direction 3

PARTICIPANT K

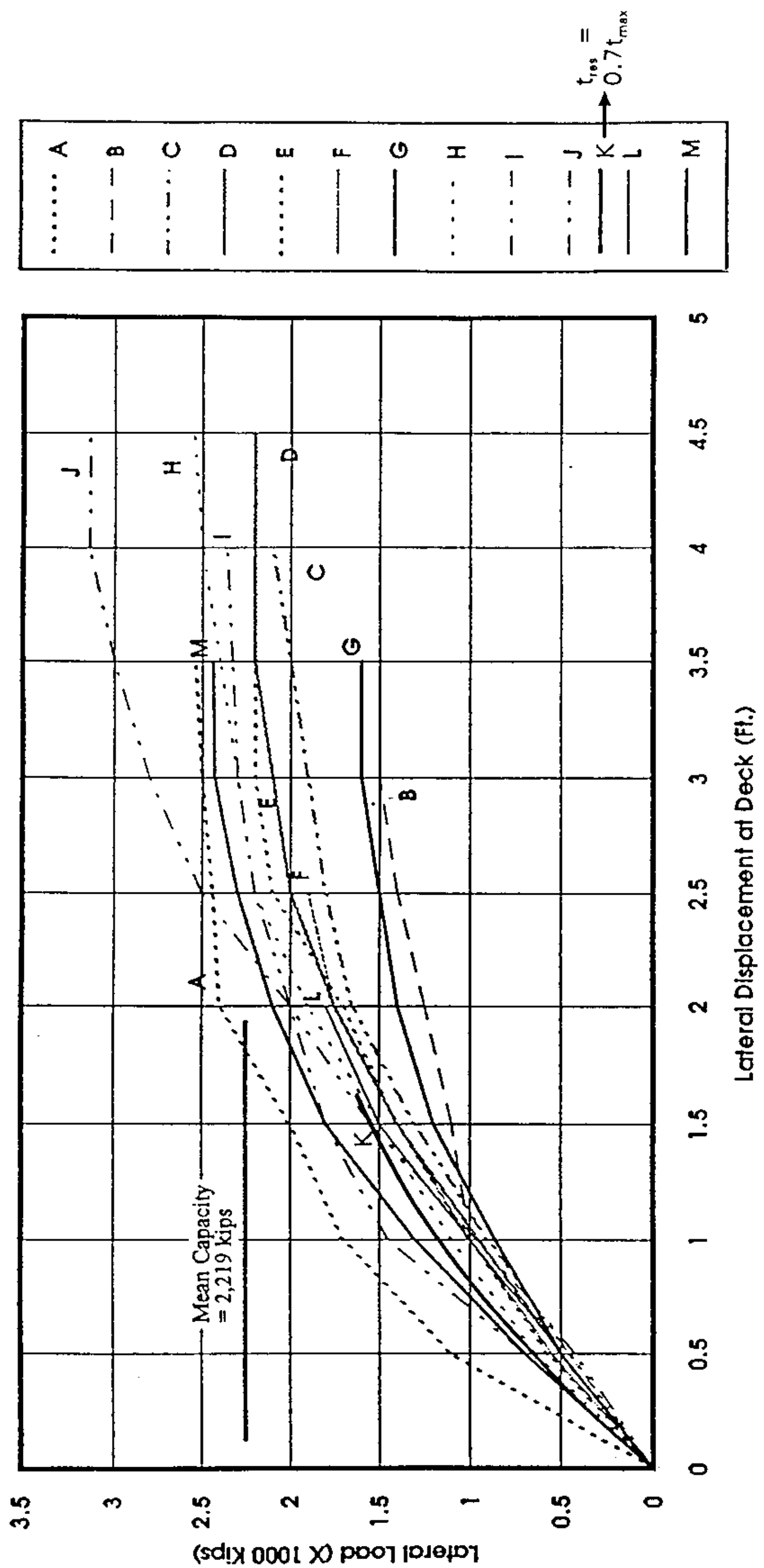

Figure 3-11 Load-Displacement Behavior - Direction 2

PARTICIPANT K

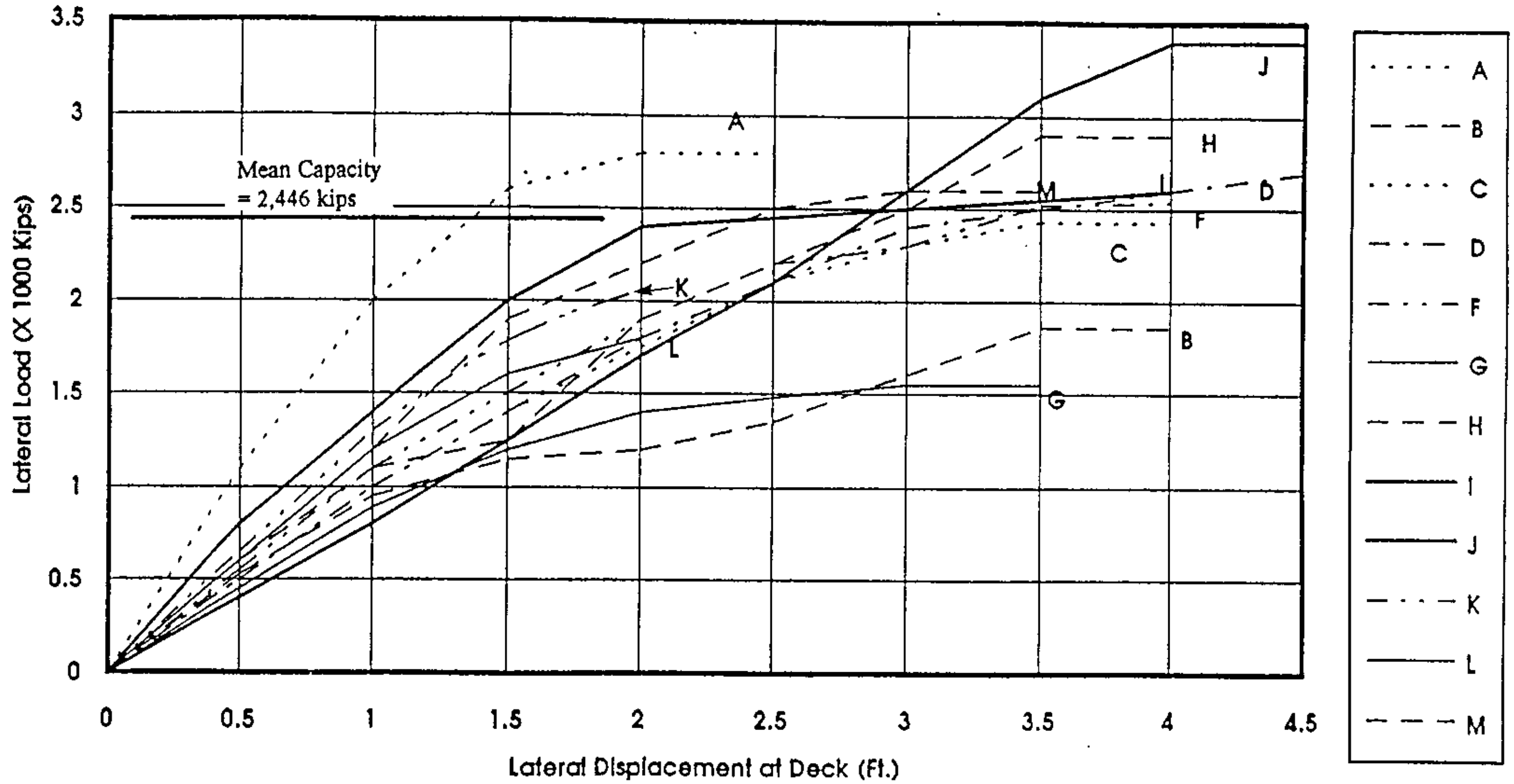

Figure 3-12 Load-Displacement Behavior - Direction 3

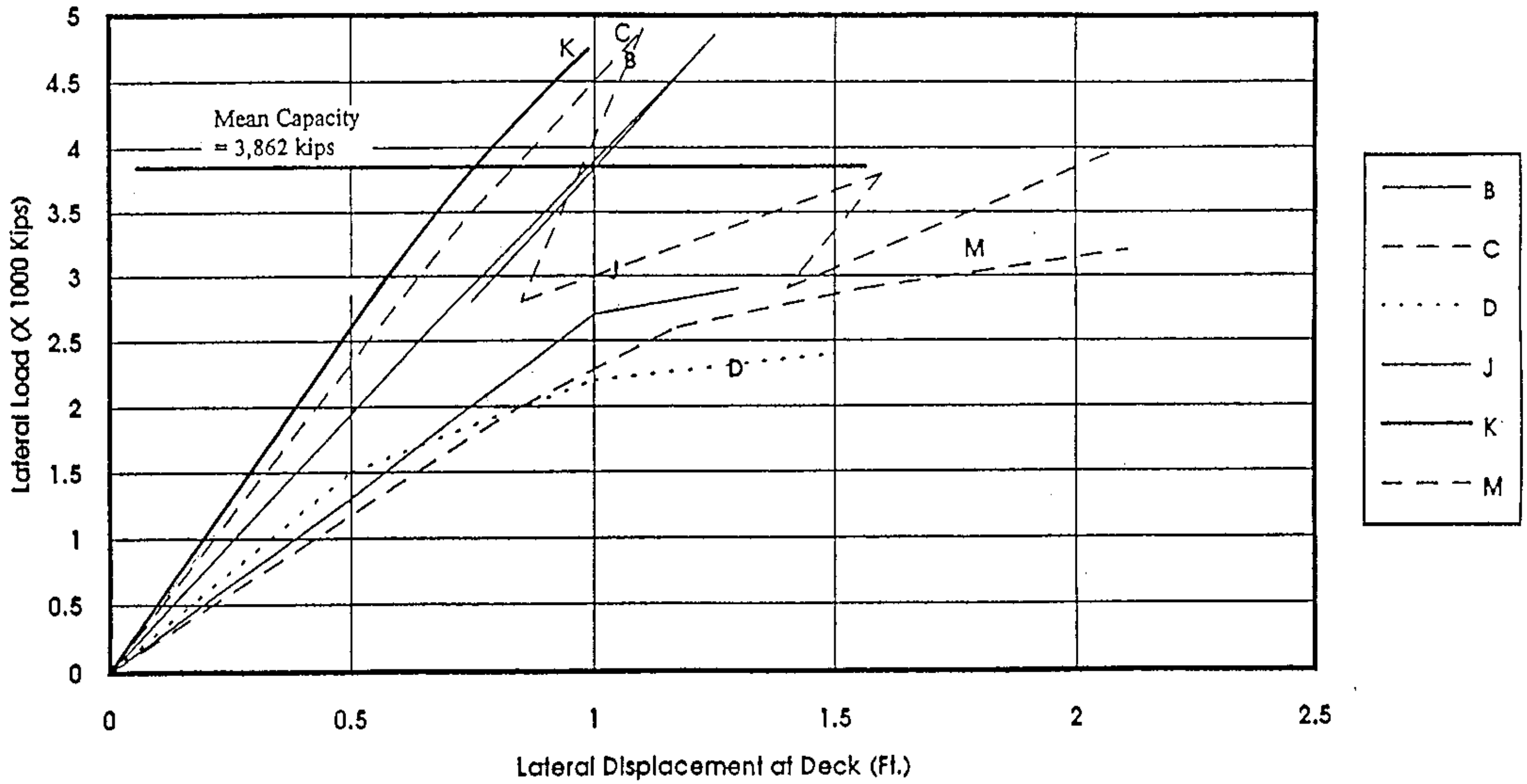

Figure 3-15 Load-Displacement Behavior - Fixed Base Case - Direction 2

PARTICIPANT K

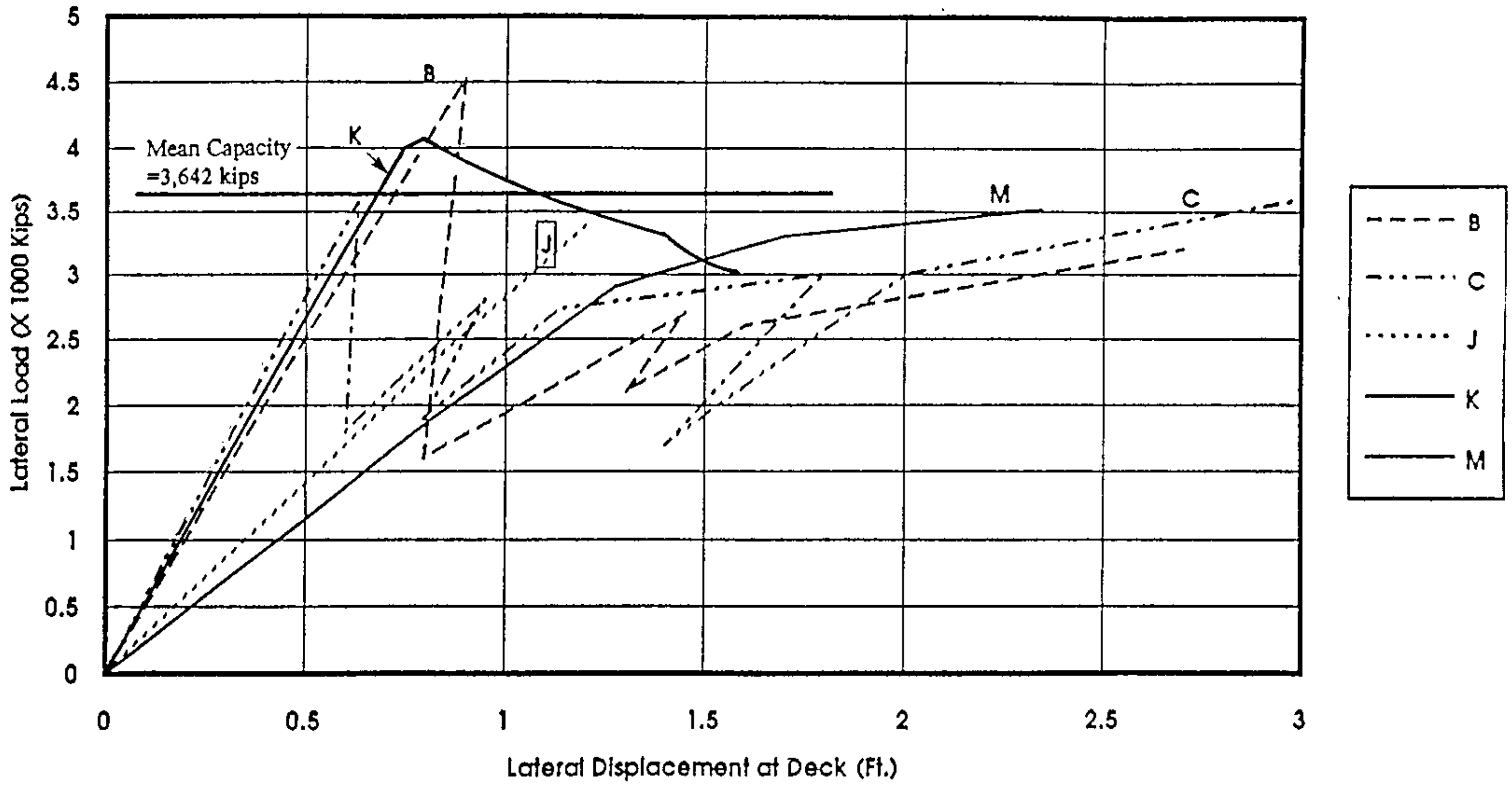

Figure 3-16 Load-Displacement Behavior - Fixed Base Case - Direction 3

Appendix B
API TG 92-5 Response to Participants' Comments

This appendix provides reponse to the participants' comments summarized in Section 4. The comments received were discussed in brief at the final meeting held on October 18, 1994. The two documents received from the API TG are given in two parts of this Appendix.

Part B.1: API TG 92-5 reply to Trial and Benchmark Participants' Comments on Section 17

Part B.2: Metocean criteria and wave/current force calculation procedures for the Gulf of Mexico PMB Trials JIP Benchmark Platform.

Appendix B.1

Part B.1: API TG 92-5 reply to Trial and Benchmark Participants' Comments on Section 17

API TG 92-5 REPLY TO
TRIAL AND BENCHMARK PARTICIPANT'S
COMMENTS ON SECTION 17

The Task Group wishes to thank all Participants for their comments and questions. Your input greatly assisted us in finalizing the Draft document and will continue to help in preparing the Final version of Section 17. Our reply is organized by Section with no separate comments for Trial or Benchmark Participants. We have tried to correct all noted typographical errors.

17.1 - GENERAL

1. Section 17 is intended to be and will be an addendum to the 20th Edition of API RP 2A.

2. Participants are directed to the noted references for details on background and philosophy. The main reason for including the complete reference title in the text for this Draft edition is to assist in guiding uses to the right document for any additional information desired.

17.2 - PLATFORM ASSESSMENT INITIATORS

1. The Draft version of Section 17 which will be published by the API shows assessment initiators in Figure 17.5.2 with a question regarding regulatory requirements. The decision to reference assessment initiators, rather than state them explicitly in the flowchart, was based upon space limitations.

2. The question of joint strength is presently being addressed by the API. It is recognized that the API joint strength formulas are in conservative. This was considered in the definition of significant contained in 17.2.6.

3. There is no "defined" significance to the words "must", "shall" and "should".

4. The wording in 17.2.6 is correct.

5. In 17.2.5 we are considering changing the first word "justified" to "judged acceptable" and the phrase "justified as" to "determined to be".

6. The comments regarding Definition of Significant for loading less than 10% which could induce failure of local elements that would in turn lead to overall failure of the platform are being considered for the final version.

17.3 - EXPOSURE CATEGORIES

1. Unmanned bridge-connected structures should not be considered manned unless their failure could be a hazard to any adjacent manned structure. A clarification is planned for the final version.

2. The word not has been added to 17.3.2a.

17.4 - PLATFORM ASSESSMENT INFORMATION - SURVEYS

1. 17.4.3. Soil Data is included here because it is felt that this is one of the important pieces of <u>information</u> required in order to perform an assessment.

17.5 - ASSESSMENT PROCESS

1. The flowchart in Figure 17.5.2 is being changed to read "Design Level Analysis"

2. The statement in 17.6.1 is more precise; however, opinions differ as to the preferred wording. There is currently a reference to Section 17.6 at the end of the first paragraph in 17.5.2.

3. The check to determine if platform damage or increased loading is significant is implicit in the "assessment initiators" diamond, which refers to section 17.2. The comments beginning, "Some analytical work is necessary," are all valid.

4. The criteria for platform assessment, and their basis, are provided in OTC 7482 (1994). For "minimum consequence" platforms, the present practice of accepting undamaged platforms (or platforms with insignificant damage) was adopted. This leads to two levels of acceptance criteria, best explained by example. Referring to Figure 4 in OTC 7482, platform L was found to have a LRF of 0.15. This is less than the 0.30 used as the basis of the US-GOM criteria, per Table 1. Nonetheless, if damage or increased loading was found to be significant relative to the as-built condition (the LRF of 0.15), as a "minimum consequence" platform it would be considered acceptable. This is why a design level analysis of both the existing and as-built structures can be of benefit--checking only against the minimum consequence criteria might result in a platform such as "L" failing the assessment, while checking relative to the "as-built" condition (i.e., performing two design level analyses) may yield the opposite conclusion.

   The advantage of setting an absolute criterion in conjunction with one relative to the as-built condition is that it avoids the following potential inconsistency. If all minimum consequence platforms had to be brought to within 10%, or some other fixed percentage of as-built platform strength, a company could be required to repair platforms that, though damaged, had higher capacity than other older and weaker (though undamaged) platforms. While this might make sense in terms of economic risk, it is not consistent with life or environmental safety assessment.

   This dual basis for acceptance (relative to the as-built condition and to an absolute criterion) is only provided for minimum consequence platforms. Due to changes in design practice over the past forty years, some manned or significant environmental impact platforms which have not suffered either damage or increased loading may fail the design level or ultimate strength assessments. It has been assumed to date that another initiator, either inadequate deck height, a regulatory requirement, or a possible "obsolescence" criterion (being considered) will initiate the assessment of such platforms.

Note: If the only initiator is damage or increased loading which cannot, a priori, be discounted as insignificant, the wording in section 17.2.6 does imply that it would simply be necessary to demonstrate that such changes were in fact "insignificant". However, the intent of those involved with developing acceptance criteria was that all manned or significant environmental impact platforms should meet the criteria in Tables 17.5.2a and b, even if there has been no change in strength from the as-built condition. This could be achieved implicitly, through the design basis check, or explicitly, through design level or ultimate strength analysis.

For increased loading, only a wave loading analysis is necessary. For assessing damage to a platform, structural analysis, at an element level up to a full structural analysis (design level or ultimate strength) is required. Wording in the final version of Section 17 will reflect this.

5. The wording suggested for clarity prior to Section 17.5.2.4 will be incorporated in the final version.

6. Regarding the design basis check; the requirement for platforms to have been designed to the 9th edition or later was based upon both the hydrodynamic loading recipe and the design equations used to ensure adequate member and joint strength. Consequently it is not sufficient just to demonstrate that a platform designed prior to 1977 meets the reference level loading in the 9th edition.

7. The word "requirements" in 17.5.2.3 and 17.5.2.4 refers to the specific requirements listed in the referred procedures (17.7.2 and 17.7.3). There are requirements and exceptions to requirements listed in these procedures.

8. As noted in 17.5.1, the screening of platforms to determine which ones should proceed to detailed analysis is performed by executing the first four components of the assessment process; platform selection, categorization, condition assessment and design basis check. For Seismic and Ice loading this is the screening criteria and is discussed in more detail in OTC 7485 (1994). Greater clarification might have been achieved with the wording "platforms that are not screened out as acceptable for seismic (or ice) loading" may be .....

   Note: Section 17.4 (part of the screening process) requires a Level II survey.

9. Regarding the question on explicit probabilities of failure:

   There are no target criteria specified, nor is there a defined scope for all failure probabilities to include (fire, blast, etc.). The language in the commentary is purposefully vague, placing the burden of justifying the adequacy of criteria upon the owner. The benchmark study has illustrated the variability that can arise when assessing platforms on the basis of estimated ultimate strength, or even based on design level analysis; attempting to provide consistency in assessments based on probability of failure could prove even more challenging. None the less, there are advantages to the probabilistic approach, and there was broad consensus to leave it as an option. However, there was also consensus that probability of failure targets should not be specified without giving extensive guidance as to how the assessment should be performed, and what assumptions are reasonable regarding uncertanity of loading and strength.

17.6 - METOCEAN, SEISMIC AND ICE CRITERIA/LOADS

1. Numerous questions regarding metocean criteria were previously addressed by TG 92-5's WG #3 with the answers sent out by PMB in a project update. Those questions and comments will not be addressed here.

2. WG #3 is separately providing the "correct" criteria that should have been used for the benchmark study. This will answer many of the questions noted.

3. Again, Participants are referred to reference #1 from BOSS '94 and OTC papers #7482 and 7485 (1994) for additional details/background on metocean criteria and RSR's.

4. Yes, an ultimate strength analysis is required if the deck height is not adequate. The design level analysis criteria was developed based on experience with structures which did not have any wave in deck loading and is only appropriate for such platforms. The ultimate strength analysis criteria is derived from experience with platforms which experienced wave in deck loading.

17.7 - STRUCTURAL ANALYSIS FOR ASSESSMENT

1. Assessment based on prior exposure is allowed for under C17.5.1.3. This method for joint capacity is only appropriate for ultimate strength analysis, not design level analysis.

2. Software validation, while an appropriate desire, is not specifically required in general and is not intended to be a requirement for assessment.

3. The method to be used for ultimate strength analysis is left up to the engineer. If there is any question in his mind as to the adequacy of liniar global analysis or local overload considerations he should proceed to global inelastic analysis. It is likely this Task Group will direct attention to being more specific in some ultimate strength modeling provisions. We felt for this time we had to be general to allow for alternative procedures. With industry experience more guidance will be available, especially in the area of joint capacity.

4. Sufficient static push-over analysis should be performed to determine the MINIMUM RSR.

5. in C17.7.3c.3.g, the phrase "(displacement generally greater than 10% of the pile diameter)" will be deleted in the final version of Section 17.

Appendix B.2

Part B.2: Metocean criteria and wave/current force calculation procedures for the Gulf of Mexico PMB Trials JIP Benchmark Platform.

Chevron

API Correspondence
TG 95-2 on Platform Assessment
WG3 - Environmental Loading

December 14, 1994

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Mr. Frank Puskar
Mr. Rajiv Aggarwal
PMB Engineering Inc.
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San Francisco, California 94111

Gentlemen:

Enclosed is a report from WG3 containing the "correct" metocean criteria and force calculation procedures for evaluating the Gulf of Mexico Trials JIP Benchmark Platform. The assessment criteria are based on API RP2A-WSD 20th Edition, Draft Section 17, Assessment of Existing Platforms, June 28, 1994. These criteria have been checked for accuracy by TG 95-2 WG3 members.

Metocean criteria and force calculation procedures are provided for each of eight principal directions (with respect to the platform). The criteria and procedures are for 20th edition design forces, and Section 17 design level and ultimate strength analyses. The method used to arrive at the criteria is described in enough detail so that the basis for the numbers would be clear.

We understand that these criteria will be used by a number of participants to recalculate the base shears in the JIP final report. WG3 asks that each participant highlight the steps where they differ from the given criteria and send comments in writing to WG3, who will then transmit the information to the Wave Force Task Group (being reinstated) for their use in clarifying the 20th edition and Section 17 wave force recipes. Specifically we would like to know what each of the participants used for: (1) wave height, (2) current, (3) storm tide, (4) wave period, (5) wind speed, (6) marine growth, (7) wave kinematics factor, (8) current blockage factor, (9) current profile, (10) drag and inertia force coefficients for both rough and smooth members, (11) wave theory, and (12) conductor shielding factor. Some of this information has been already provided in the JIP report. The Wave Force Task Group would like to receive all pertinent information.

Although there is room for specifying differing criteria because of misinterpretation of intent and acceptable range on parameter values, the effect on base shear should be small and would not result in the large range of base shears that resulted in the JIP. Nevertheless some improvements can be made towards clarification of the procedures. With your information, the Wave Force Task Group will amend the text of the 20th edition and Section 17 to provide for, hopefully, more uniform results on base shear when different personnel use the documents.

Very truly yours,

C. Petrauskas

C. Petrauskas
Team Leader, WG3 of TG 95-2

enc: As noted above

cc w/enc:

WG3 Members
Jim Bole (Amoco, Tulsa)
Kris Digre (Shell Offshore, Houston)
Allan Reece (Shell Development, Houston
Roger Thomas (Phillips, Bartlesville)
Dave Wisch (Texaco, Bellaire)

METOCEAN CRITERIA AND WAVE/CURRENT FORCE CALCULATION PROCEDURES THE GULF OF MEXICO PMB TRIALS JIP BENCHMARK PLATFORM

C. Petrauskas (for WG3 of TG 95-2)

Tue, Dec 6, 1994

INTRODUCTION

The platform is located in the Gulf of Mexico at 28° 27' N latitude and 91° 20' W longitude (Ref 1). The water depth at the platform location is 157 ft (Ref 2). The platform has four legs, and is oriented so that the diagonal directions are north/south and east/west. Various analyses were required by the Trials JIP using the API RP2A WSD 20th ed wave forces (Ref 3).

This report defines the appropriate metocean criteria and wave force calculation procedures to arrive at the platform base shears that are consistent with the intent of (a) the guideline 20th edition design forces in Ref 3 and (b) Section 17 design level and ultimate strength significant-environmental-impact forces in Ref 4. The results are given for all eight principal platform directions, although only three principal platform directions (Fig 1) were used by most participants.

API RP2A-WSD 20TH EDITION CRITERIA AND FORCES

Wave Heights

The platform is located in a region for which 20th ed metocean criteria are applicable (Ref 3, Fig 2.3.4-2). The water depth is assumed to be equal to Mean-Lower-Low-Water (MLLW). The omnidirectional wave height is 63 ft (Ref 3, Fig 2.3.4-3).

Wave heights, as a function of the required (for force calculations) wave direction, are given in Table 1, column 2. The wave heights were obtained by using the guideline design factors given in Ref 3, Fig 2.3.4-4, and taking into account that the factors apply to the guideline design direction ± 22.5° (Ref 3, Sec 2.3.4c3). Interpolation should not be used.

Storm Tide

The storm tide is 3.5 ft (Ref 3, Fig 2.3.4-7) for all directions. This is the sum of storm surge and astronomical tide. The storm water depth for the benchmark platform is 160.5 ft (157 ft + 3.5 ft).

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 longitude. The depth of 160.5 ft places the current in the "Intermediate Zone" (Ref 3, Sec 2.3.4c4). 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 Ref 3, p. 123, "Commentary on Hydrodynamic Force Guidelines, Section 2.3.4". Note that the example only provides the steps for a wave direction of 290°. Such an interpolation has to be carried for all eight required directions given in Table 1. From a practical point of view, the 160.5 ft water depth is sufficiently close to the depth of 150 ft at the shallow-water-zone/intermediate zone boundary, that interpolation may not be necessary. However, for completeness, the interpolation was carried out for all eight directions.

"Shallow Water Zone" Current

The longitude of the platform is 91.33°. The surface current is a vector with a magnitude of 2.1 kts (3.55 fps). Its direction, based on Ref 3, Fig 2.3.4-5, is 280°. For interpolation, the water depth is taken as 150 ft.

"Deep Water Zone" Current

In deep water only the component of the current in the direction of the wave is important, the transverse current is negligible. According to Ref 3, Sec 2.3.4c4 the magnitude of the surface current in the principal wave direction (290°) is 2.1 kts. The magnitudes of the current for the rest of the wave directions, given in Ref 3, Fig 2.3.4-4, are obtained by applying, to the 290° current, the same factor that is applied to the wave heights. This current is assumed to apply to the given direction ± 22.5°. For interpolation, the water depth is taken as 300 ft.

Interpolated Current at Platform Location

The interpolated inline and transverse currents for a water depth of 160.5 ft is given in Table 1, columns 3 and 4, respectively. A negative inline current means that the inline component of the current opposes the wave. A negative transverse current is the transverse component that is directed clockwise with respect to the inline component.

In performing the interpolation we noted that the example in the Commentary is not consistent with the intent in the main text. Specifically, the check on whether or not the inline current is ≥ 0.2 kts 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 ed.

Current for Design Guideline Forces

The appropriate surface current for calculating the 20th ed design guideline forces is given in Table 1, column 5. This is the same as the inline current in column 3, except it is modified to make sure that the speed is ≥ 0.2 kts (see Ref 3, Sec 2.3.4c4). The current profile is uniform over the water column (Ref 3, Fig 2.3.4-6).

The author believes that it is sufficient to use the inline current for analysis. However it is acceptable to include the transverse component of the current, given in column 4, provided the specified vector current is consistent with the inline component given in column 5. This issue will receive further attention by the API Task Group on Wave Force Commentary and a clarification will be provided for the 21st ed of RP2A.

Wave Periods

The wave period is 13 sec for all directions (Ref 3, Section 2.3.4c5). This is the period measured at a fixed point. For the purpose of obtaining wave kinematics that may be superimposed on the inline current, the apparent wave period (Tapp, period measured in a coordinate system with the wave) is needed. Tapp is given in Table 1, column 8. It is based on the inline current in column 5 and is calculated using Ref 3, Fig 2.3.1-2.

Wind Speed

The one-hour wind speed at an elevation of 10 m is 80 knots (Ref 3, Section 2.3.4c7).

Marine Growth

The thickness is 1.5" and extends from +1 ft to -150 ft (Ref 3, Sec 2.3.4d2).

Wave Kinematics Factor

For hurricanes the wave kinematics factor is 0.88 (Ref 3, Sec 2.3.4d1).

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 (Ref 3, Sec 2.3.1b4). The blockage factor should be applied to the inline current given in Table 1, column 5.

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, these directions may not control the design.

A simple measure of whether or not default values are applicable is Umo*Tapp/D, where Umo is the maximum horizontal velocity at storm water level and D is the diameter of platform leg at the storm water level (see Ref 3, Sec 2.3.1b7). If Umo*Tapp/D $\geq$ 30, default values apply; otherwise one needs to consult the Commentary for appropriate coefficients. Default values of the coefficients are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2.

Wave Theory

The appropriate wave theory should be selected from Ref 3, Fig 2.3.1-3. Other wave theories such as Extended Velocity Potential and Chappelear may be used if an appropriate order of solution is selected.

Conductor Shielding Factor

Ignore shielding (shielding factor = 1.0) because there are only four conductors and the spacing is irregular.

SECTION 17 SIGNIFICANT ENVIRONMENTAL IMPACT CRITERIA AND FORCES

Design Level Wave Heights

The omnidirectional wave height is 55 ft (Ref 4, Fig 17.6.2-2a).

Wave heights, as a function of the required (for force calculations) wave direction, are given in Table 2, column 2. The wave heights were obtained by choosing, for each direction, the lower value of the 55-ft wave height vs the 20th ed wave height.

Design Level Storm Tide

The storm tide is 3.0 ft (Ref 4, Fig 17.6.2-2a) for all directions. This is the sum of storm surge and astronomical tide. The storm water depth is 160 ft (157 ft + 3.0 ft).

Design Level Current

The appropriate surface current is given in Table 2, column 5. The currents were obtained by choosing, for each direction, the lower value of 1.6 kts (Ref 4, Table 17.6.2-1) vs the 20th ed current.

The current profile is uniform over the water column (Ref 3, Fig 2.3.4-6).

Design Level Wave Periods

Tapp is given in Table 2, column 8. It is based on the inline current in column 5 and is calculated using Ref 3, Fig 2.3.1-2.

Design Level Wind Speed

The one-hour wind speed at an elevation of 10 m is 65 knots (Ref 4, Table 17.6.2-1).

Design Level Marine Growth

The thickness is 1.5" and extends from +1 ft to -150 ft (Ref 3, Sec 2.3.4d2).

Design Level Wave Kinematics Factor

For hurricanes the wave kinematics factor is 0.88 (Ref 3, Sec 2.3.4d1).

Design Level 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 (Ref 3, Sec 2.3.1b4). The blockage factor should be applied to the inline current given in Table 2, column 5.

Design Level Force Coefficients

For the Trials JIP benchmark platform it is assumed that default values of force coefficients apply for all load cases. The default values are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2.

The applicability of default values will be further addressed by the API Task Group on Wave Force Commentary and a clarification will be provided for the 21st ed of RP2A.

Design Level Wave Theory

The appropriate wave theory should be selected from Ref 3, Fig 2.3.1-3. Other wave theories such as Extended Velocity Potential and Chappelear may be used if an appropriate order of solution is selected.

Design Level Conductor Shielding Factor

Ignore shielding (shielding factor = 1.0) because there are only four conductors and the spacing is irregular.

Ultimate Strength Wave Heights

The omnidirectional wave height is 68 ft (Ref 4, Fig 17.6.2-2a).

Wave heights, as a function of the required (for force calculations) wave direction, are given in Table 3, column 2. The wave heights were obtained by applying the same factors that were applied to arrive at the 20th ed wave heights.

Ultimate Strength Storm Tide

The storm tide is 3.0 ft (Ref 4, Fig 17.6.2-2a) for all directions. This is the sum of storm surge and astronomical tide. The storm water depth is 160 ft (157 ft + 3.0 ft).

Ultimate Strength Current

The appropriate surface current is given in Table 3, column 5. The currents were obtained using the same procedure that was used for the 20th ed currents. The current magnitude is 2.3 kts (Ref 4, Table 17.6.2-1) as opposed to the 2.1 kts for the 20th ed.

The current profile is uniform over the water column (Ref 3, Fig 2.3.4-6).

Ultimate Strength Wave Periods

Tapp is given in Table 3, column 8. It is based on the inline current in column 5 and is calculated using Ref 3, Fig 2.3.1-2.

Ultimate Strength Wind Speed

The one-hour wind speed at an elevation of 10 m is 65 knots (Ref 4, Table 17.6.2-1).

Ultimate Strength Marine Growth

The thickness is 1.5" and extends from +1 ft to -150 ft (Ref 3, Sec 2.3.4d2).

Ultimate Strength Wave Kinematics Factor

For hurricanes the wave kinematics factor is 0.88 (Ref 3, Sec 2.3.4d1).

Ultimate Strength 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 (Ref 3, Sec 2.3.1b4). The blockage factor should be applied to the inline current given in Table 2, column 5.

Ultimate Strength Force Coefficients

For the Trials JIP benchmark platform, it is assumed that default values of force coefficients apply for all load cases. The default values are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2.

The applicability of default values will be further addressed by the API Task Group on Wave Force Commentary and a clarification will be provided for the 21st ed of RP2A.

Ultimate Strength Wave Theory

The appropriate wave theory should be selected from Ref 3, Fig 2.3.1-3. Other wave theories such as Extended Velocity Potential and Chappelear may be used if an appropriate order of solution is selected.

Ultimate Strength Conductor Shielding Factor

Ignore shielding (shielding factor = 1.0) because there are only four conductors and the spacing is irregular.

REFERENCES

1. PMB Engineering, Trials JIP, Benchmark Analysis, Revision 2, April 12, 1994

2. PMB Engineering, Trials JIP, Benchmark Analysis, Draft Final Report, September, 1994

3. American Petroleum Institute, Recommended Practice 2A-WSD (RP 2A-WSD), Twentieth Edition, July 1, 1993

4. American Petroleum Institute, Recommended Practice 2A-WSD (RP 2A-WSD), Twentieth Edition, Draft Section 17, Assessment of Existing Platforms, June 28, 1994

Tables 1 - 3
Figures 1 - 2

TABLE 1 (revised 11Dec94)

Guideline Design Metocean and Wave Force Criteria for Gulf of Mexico
Benchmark Platform, MLLW=157', Static Analysis, 20th Ed API RP2A

1	2	3	4	5	6	7	8	9
Wave Dir (deg. towards, clockwise from North)	Wave Height (ft)	Inline Current (kts)	Transverse Current (kts)	Inline Current (kts)	Storm Tide (ft)	Wave Period (sec)	Apparent Wave Period (sec)	Wind Speed (1-hr@10m) (kts)
90.0	44.1	-1.82	0.34	0.20	3.5	13.0	13.1	80.0
45.0	44.1	-1.02	1.60	0.20	3.5	13.0	13.1	80.0
0.0	53.6	0.46	1.92	0.46	3.5	13.0	13.2	80.0
315.0	59.9	1.74	1.12	1.74	3.5	13.0	13.7	80.0
270.0	63.0	2.07	-0.34	2.07	3.5	13.0	13.8	80.0
225.0	56.7	1.25	-1.60	1.25	3.5	13.0	13.5	80.0
180.0	47.3	-0.23	-1.92	0.20	3.5	13.0	13.1	80.0
135.0	44.1	-1.50	-1.12	0.20	3.5	13.0	13.1	80.0

Marine Growth	Thickness = 1.5" (from + 1.0 ft to -150.0 ft)
Wave Kin. Factor	0.88
Current Blockage Factor	0.80 for end-on and tranverse directions 0.85 for diagonal directions
Current Profile	Uniform over the water column
Force Coeff.	If Umo*Tapp/D ≥ 30 use default values, otherwise consult Commentary Umo = maximum horizontal velocity at storm water level Tapp = apparent wave period D = platform leg diameter at storm water level Default values are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2
Wave Theory	Select wave theory from Fig. 2.3.1-3, or use appropriate order of other equivalent theory, such as Chappelear or Velocity Potential
Conductor Shielding Factor	Use 1.0 because there are only four conductors and the spacing is irregular

TABLE 2 (revised 11Dec94)

Significant Environmental Impact Design Level Metocean and Wave Force Criteria for Gulf of Mexico Benchmark Platform, MLLW=157', Static Analysis

1	2	3	4	5	6	7	8	9
Wave Dir (deg. towards, clockwise from North)	Wave Height (ft)	Inline Current (kts)	Transverse Current (kts)	Inline Current (kts)	Storm Tide (ft)	Wave Period (sec)	Apparent Wave Period (sec)	Wind Speed (1-hr@10m) (kts)
90.0	44.1	1.6	NA	0.20	3.0	12.1	12.2	65.0
45.0	44.1	1.6	NA	0.20	3.0	12.1	12.2	65.0
0.0	53.6	1.6	NA	0.46	3.0	12.1	12.3	65.0
315.0	55.0	1.6	NA	1.60	3.0	12.1	12.6	65.0
270.0	55.0	1.6	NA	1.60	3.0	12.1	12.6	65.0
225.0	55.0	1.6	NA	1.25	3.0	12.1	12.5	65.0
180.0	47.3	1.6	NA	0.20	3.0	12.1	12.2	65.0
135.0	44.1	1.6	NA	0.20	3.0	12.1	12.2	65.0

Marine Growth	Thickness = 1.5" (from + 1.0 ft to -150.0 ft)
Wave Kin. Factor	0.88
Current Blockage Factor	0.80 for end-on and tranverse directions 0.85 for diagonal directions
Current Profile	Uniform over the water column
Force Coeff.	Use default values Default values are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2
Wave Theory	Select wave theory from Fig. 2.3.1-3, or use appropriate order of other equivalent theory, such as Chappelear or Velocity Potential
Conductor Shielding Factor	Use 1.0 because there are only four conductors and the spacing is irregular

TABLE 3 (revised 11Dec94)

Significant Environmental Impact Ultimate Strength Metocean and Wave Force Criteria for Gulf of Mexico Benchmark Platform, MLLW=157', Static Analysis

1	2	3	4	5	6	7	8	9
Wave Dir (deg. towards, clockwise from North)	Wave Height (ft)	Inline Current (kts)	Transverse Current (kts)	Inline Current (kts)	Storm Tide (ft)	Wave Period (sec)	Apparent Wave Period (sec)	Wind Speed (1-hr@10m) (kts)
90.0	47.6	-2.01	0.37	0.20	3.0	13.5	13.6	85.0
45.0	47.6	-1.12	1.76	0.20	3.0	13.5	13.6	85.0
0.0	57.8	0.50	2.11	0.50	3.0	13.5	13.7	85.0
315.0	64.6	1.90	1.23	1.90	3.0	13.5	14.2	85.0
270.0	68.0	2.27	-0.37	2.27	3.0	13.5	14.4	85.0
225.0	61.2	1.37	-1.76	1.37	3.0	13.5	14.0	85.0
180.0	51.0	-0.26	-2.11	0.20	3.0	13.5	13.6	85.0
135.0	47.6	-1.65	-1.23	0.20	3.0	13.5	13.6	85.0

Marine Growth	Thickness = 1.5" (from + 1.0 ft to -150.0 ft)
Wave Kin. Factor	0.88
Current Blockage Factor	0.80 for end-on and tranverse directions 0.85 for diagonal directions
Current Profile	Uniform over the water colum
Force Coeff.	Use default values Default values are: Cd(smooth) = 0.65, Cd(rough) = 1.05, Cm(smooth) = 1.6, and Cm(rough) = 1.2
Wave Theory	Select wave theory from Fig. 2.3.1-3, or use appropriate order of other equivalent theory, such as Chappelear or Velocity Potential
Conductor Shielding Factor	Use 1.0 because there are only four conductors and the spacing is irregular

Appendix C
Supplemental Data from Participants

Part C.1: PMB letter dated October 25, 1994

Part C.2: Response from participants (only selected ones of relevance to Appendix A provided here)

Appendix C.1

Part C.1: PMB letter dated October 25, 1994.

MB
INEERING

500 Sansome Street
San Francisco, CA 94111

FAX 415 986 2699
TELEX 278058

October 25, 1994

Attention:

Subject: Trials JIP - Final Meeting Minutes

Gentlemen:

The final meetings for the Trials JIP project were held as scheduled on October 18 and 19 at PMB/Bechtel Houston offices and were attended by thirty four persons representing various companies. Copies of the lists of attendees are enclosed.

The attendees were provided with the meeting handouts. These handouts are also attached here for those Technical Advisory Committee (TAC) members who did not attend the meeting.

The following items were agreed upon by a majority of the TAC during the meetings.

Trial Applications

Further Submittals

The two participants who have not submitted their Trial Application Reports shall submit their reports by **November 1** to PMB. These two participants are also required to summarize their reports in the format of the draft final report. They can mark-up, by hand, all applicable tables.

Information/Action Required of Participants

All participants are required to provide the following to PMB by **November 15**:

- Any comments regarding PMB's draft report
- Where possible, the participants are requested to provide marked-up copies of the applicable tables or figures from the draft final report

The participants who have not summarized their results per the requirements of the JIP are required to submit all missing information by November 15. Specifically, this includes all information that is missing (indicated as a "?") from the tables in the PMB draft report.

Those participants who have not submitted their ultimate capacity analysis shall submit their results to PMB by November 15. These participants are also required to summarize their information in the format of the PMB draft final report. They can mark-up, by hand, all applicable tables.

Direction Provided to and Action Required of PMB

The TAC voted that the final report should include abbreviated copies of all participant submittals including the following modifications:

- All references to company names and identification will be removed from the report covers. PMB will also attempt to remove all references to company names included throughout the reports. Any references to software used for the analysis will remain as is.

- Only key figures of platforms will be included. These are platform orientation, typical vertical and horizontal framings, pile drawings, soil shear strength profiles, typical deck details.

- Result plots (which do not impact interpretation of results) and all computer outputs will be excluded.

PMB is to release the final report on **December 15, 1994**. Participants who do not provide documents meeting the minimum requirements for participation in this JIP will not receive the final report.

Benchmark Analysis

Information/Action Required of Participants

All participants are required to provide the following to PMB by **November 15**:

- Any comments regarding the draft report.

- Identification of any errors or omissions made by PMB in the summary of their report data. In such cases, the participants are requested to provide marked-up copies of the applicable tables or figures from the draft final report.

- Participants who have noted "Gross Errors" in their interpretation of the Draft Section 17 or in their analysis are requested to submit a letter to PMB identifying the errors. In these cases, participants are required to correct those tables in the draft final report that are affected by their error. PMB will expand the summary tables to reflect the elimination of all gross errors. Participants may also submit a revised document of their analysis. All re-submittals will be included as an appendix to the final report and will be included as late submittals.

  Participants are requested to identify the reasons for any "Gross Errors" or misinterpretations. Such information will be compiled by PMB and incorporated in a new Appendix.

A list of some specific questions asked during the meeting is given below for which your response is solicited. This information will help participants more fully understand your results.

<u>Base case – with pile/soil effect considered</u>

- Wave-in-deck loading estimates?
  Wave crest elevation used.

- How the conductors were modeled?
  Were conductors modeled to contribute to foundation capacity?

- Load level at first member with I.R. of 1.0. The member(s) shall be identified.
  What increase in allowable stresses were considered in the computation of I.R.?

<u>Fixed Base Case – without pile/soil effect considered</u>

- How the fixity was incorporated into the model?

<u>Direction Provided to and Action Required of PMB</u>

The TAC voted that the final report will include complete copies of all participant submittals including the following modifications:

- All references to company names and identification will be removed from the report covers. PMB will also attempt to remove references to company names included throughout the reports. Any references to software used for the analysis will remain as is.

PMB is to release the Final Report on **December 15, 1994**.

At the suggestion of Mr. Kris Digre, Chairman of the API TG responsible for the Draft Section 17 document, the TAC voted for publishing some of the results of Trial Applications and Benchmark Analyses in OTC 1995. The information would be contained in a paper that also summarizes several of the improvements to Section 17 proposed by the TG. The paper will be co-authored by several members of the TG and PMB. The information from the Project will be sanitized for confidentiality and copies of the Draft paper will be provided to the TAC prior to submittal to OTC.

All modifications or revised submittals shall be faxed or mailed to Rajiv Aggarwal in San Francisco (Fax No: 415-986-2699).

If you have any questions, please contact me at (713) 235-2918 or Rajiv Aggarwal at (415) 288-6829.

Very truly yours,

PMB ENGINEERING, INC.

Frank J. Puskar

FJP:mjw

Appendix C.2

Part C.2: Response from participants (only selected ones of relevance to Appendix A provided here).

PARTICIPANT A

JIP - TRIAL APPLICATION OF API RP 2A SECTION 17
BENCHMARK ANALYSIS

DATA RESUBMITTAL - PARTICIPANT "A"

1. SUMMARY

On August 10, 1994 Participant "A" submitted its initial benchmark analysis results to the JIP. This initial analysis effort was an optional activity to the Trials JIP and was performed at Participant's expense on a part-time basis. Participant's overall workload was very high during the time of the benchmark analysis, increasing as the due date for the initial submittal drew near. Therefore, Participant did not have the time or resources to devote a full quality effort to the benchmark analysis and a number of errors and misinterpretations were made. This resubmittal of Participant's analysis is intended to detail the errors and misinterpretations of the initial submittal and revise the affected results.

Participant "A" reviewed its initial analysis and performed the benchmark analysis a second time for two of the three analyzed storm directions. The review showed that "gross errors" were made in the initial work, most relating to input into the analysis model. The second analysis results conform to those of the other Participant's.

2. INITIAL ANALYSIS ERRORS

Several errors were made by Participant in the initial analysis. The significant errors were as follows:

- API RP 2A Section 17 Criteria

    - The "insignificant environmental impact" environmental criterion was used in the initial analysis rather than "significant environmental impact" criterion as requested in the benchmark platform description. This was an inadvertent oversight on Participant's part and is the reason for the low wave height and differing storm directions used in the ultimate load generation relative to the other participants. As a result of this error and the relative difficulty in interpreting the Section 17 and 20th Edition metocean criteria recipes, the software has now been enhanced to allow output of metocean criteria as a function of platform location, water depth, environmental criteria and life safety criteria. While this enhancement will not ensure elimination of the error made in Participant's initial analysis, it will provide for consistent analysis criteria from one analysis to another in the future.

- Soil Modeling

    - The pile-soil axial load (t-z) data was incorrectly input into the software creating an almost rigid axial stiffness of the pile-soil element. This is the major cause of the high initial stiffness of Participant "A" in the draft report load-displacement response curves. There was also a slight error in the development of the p-y curves per the API RP 2A procedures.

- Modeling Assumptions

    - Our initial report stated that the piles were assumed grouted to the legs. This statement was in error. In fact, this assumption was not incorporated into the initial nor the second analysis. The increased stiffness of our initially reported platform response was due to the soil modeling error discussed above.

3. REVISED BENCHMARK ANALYSIS RESULTS

The draft report tables have been marked-up to reflect revisions applicable to Participant "A." The marked-up tables follow.

4. OTHER SOLICITED RESULTS

A request soliciting response on various topics was issued. Our results, where available, are as follows:

- <u>Base case - with pile/soil effect considered</u>

- Wave in deck loading estimates? Wave crest elevation used?

    The attached marked-up tables present our results for wave-in-deck load. The tabulated results represent wave load on the platform structure above Elevation 16'-0. These in-deck wave loads and the associated maximum wave crest elevations are as follows:

	Wave-In-Deck Load (kips)	Max. Wave Crest Elev. (feet)
Direction 1	45	+38.2
Direction 2	236	+42.7
Direction 3	126	+36.5

- How were the conductors modeled? Were conductors modelled to contribute to foundation capacity?

The conductors were modeled as 48 or 30 inch diameter tubulars, 5/8 inch thick. The full length from conductor tip to conductor top was modeled. Soil response on the conductors was included; thus, the conductors were modeled to contribute to foundation capacity.

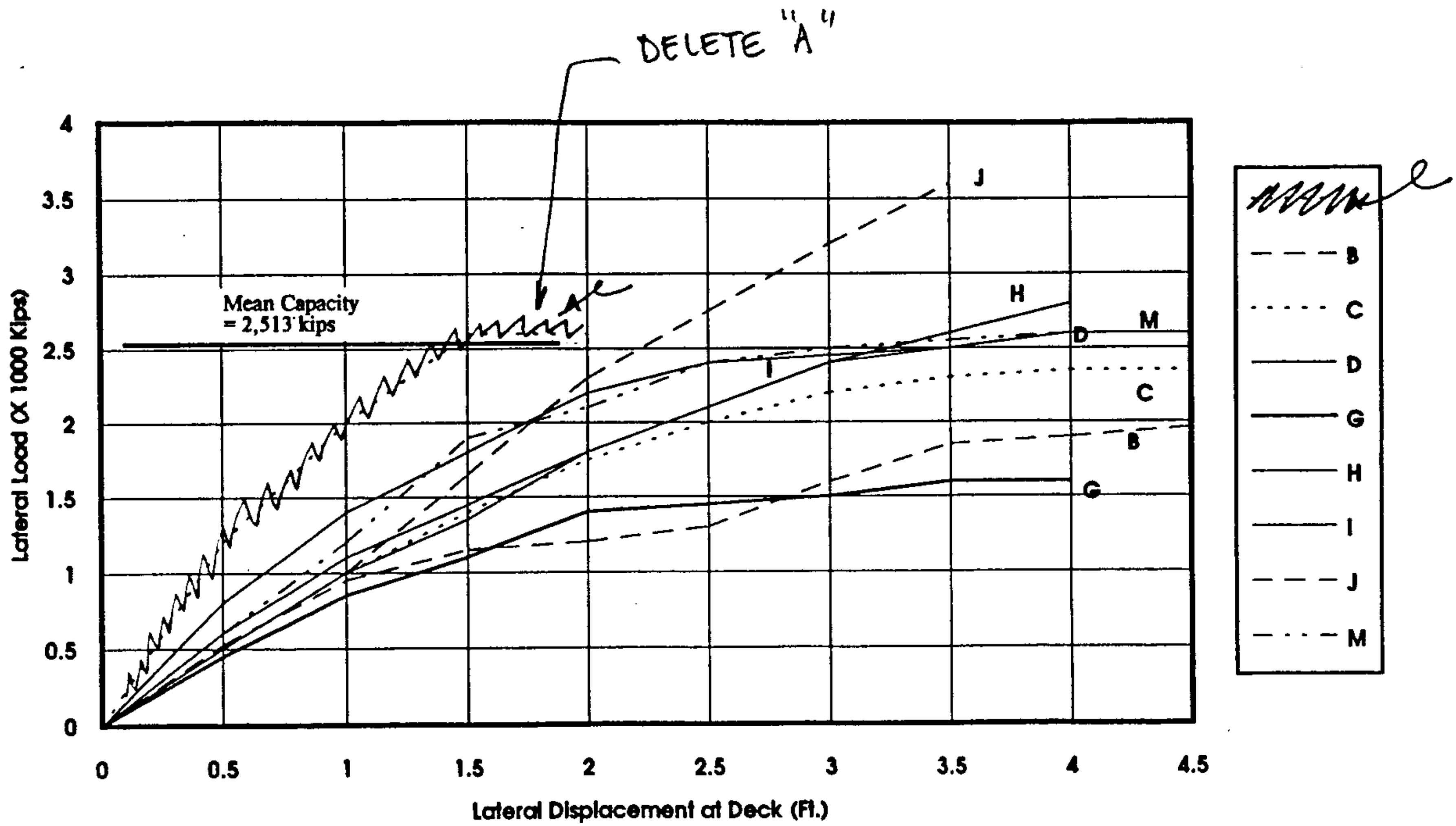


Figure 3-10 Load-Displacement Behavior - Direction 1

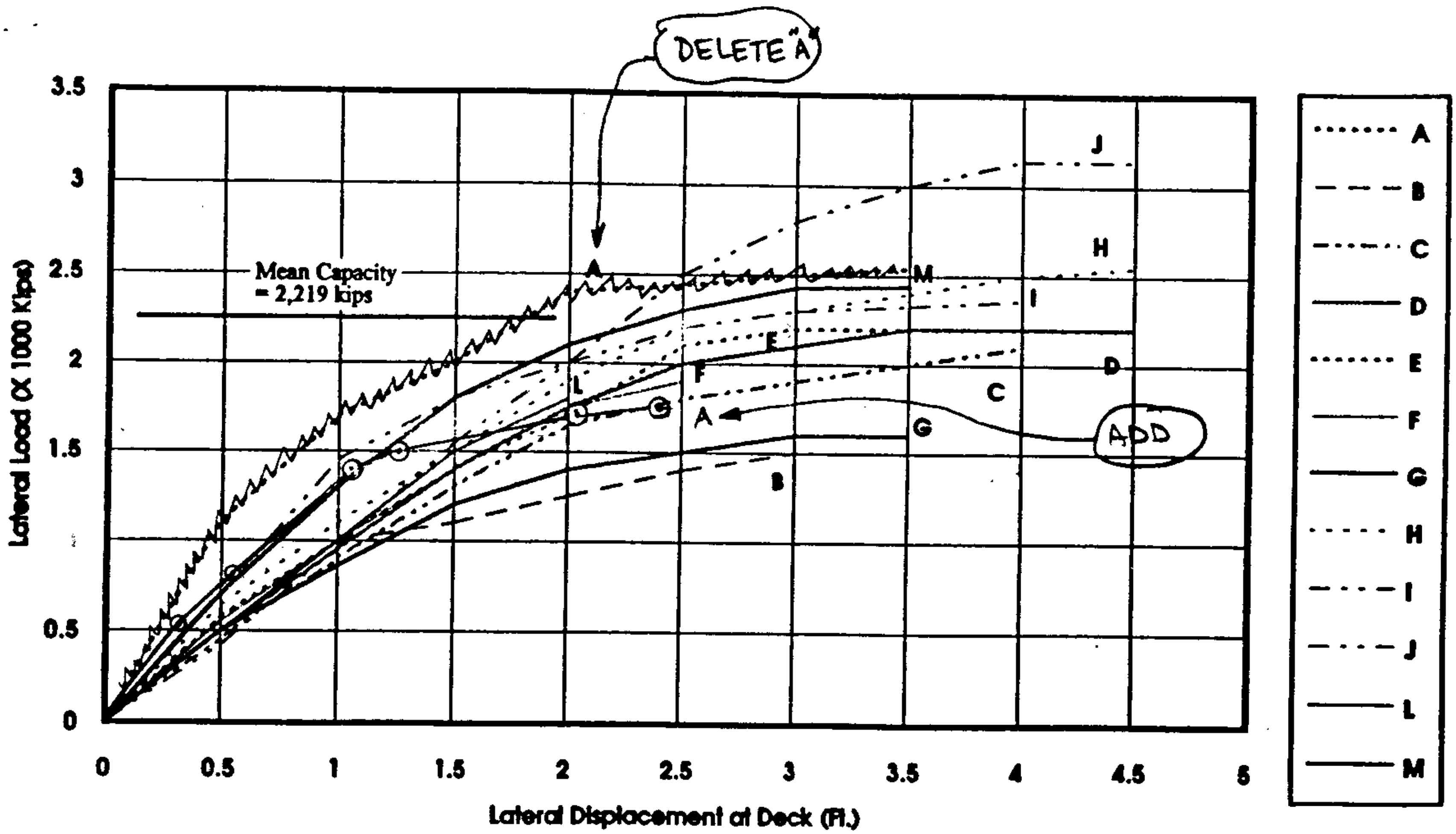
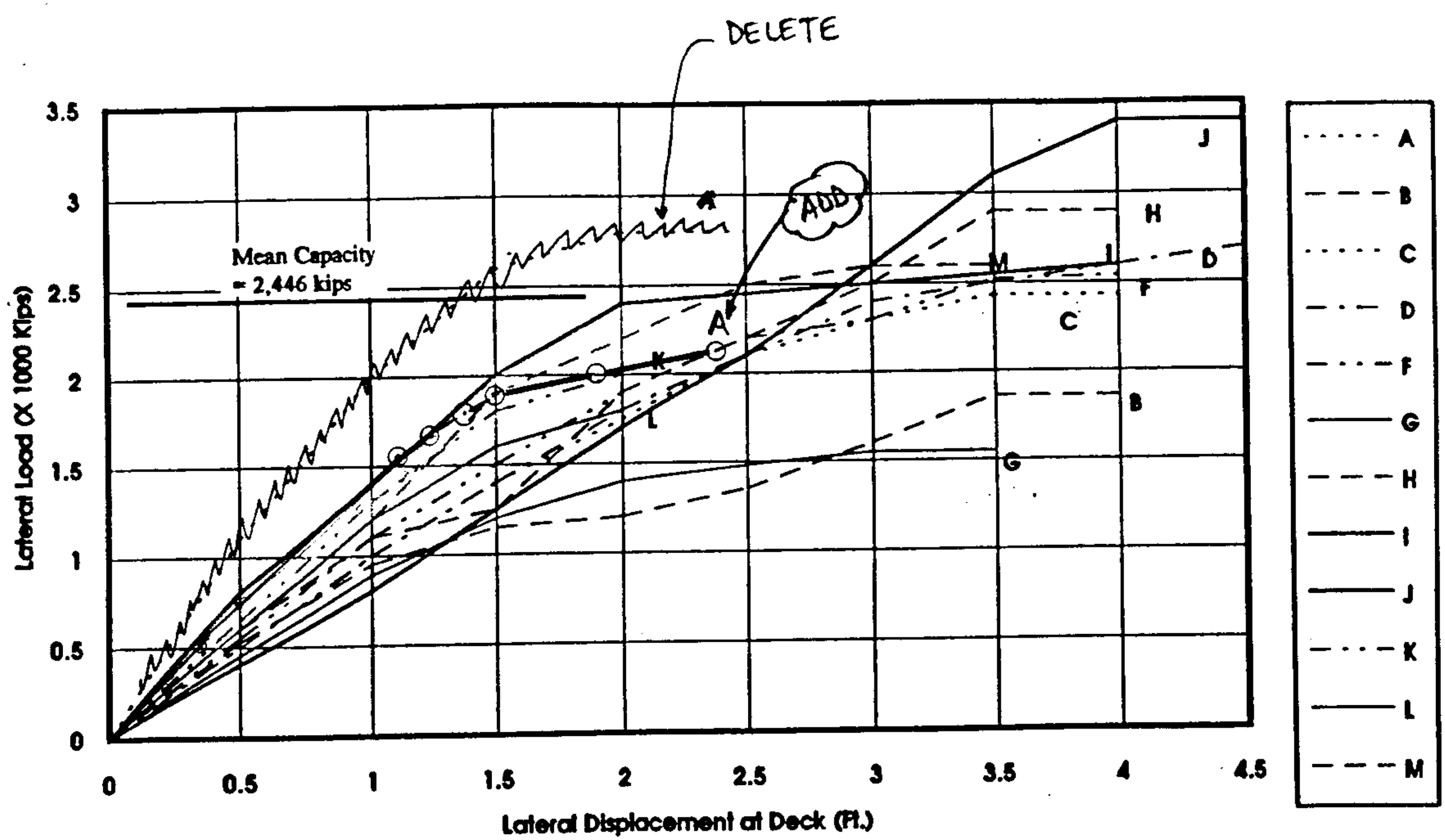


Figure 3-11 Load-Displacement Behavior - Direction 2



NOTE: SEE ATTACHED SHEET FOR DATA POINTS TO PLOT.

Figure 3-12 Load-Displacement Behavior - Direction 3

PARTICIPANT "A" DATA POINTS FOR PLOTS
(Draft Report Figures 3-11 thru 3-12)

Direction 2		Direction 3	
Lat. Load (kips)	Lat. Disp. (feet)	Lat. Load (kips)	Lat. Disp. (feet)
145	0.07	120	0.06
260	0.16	235	0.14
385	0.24	350	0.22
510	0.33	465	0.30
635	0.44	580	0.38
1,375	1.06	1,375	0.93
1,500	1.24	1,545	1.11
1,685	2.05	1,660	1.23
1,750	2.30	1,775	1.36
1,750	3.23	1,890	1.50
		2,005	1.91
		2,120	2.36

PARTICIPANT B

Corrections to Previous Results.

Please note that the current values presented earlier in the following tables were based on a primary current direction of 255° (TN) corresponding to a longitude of 90° 40′. The correct longitude should have been 91° 20′ which corresponds to a primary current direction of 273° (TN). The current values presented here are based on the latter primary direction. Old numbers are given in ( ) for comparison.

Table A.1.2: Section 17 Ultimate Strength Environmental Criteria

Wav Dir	W.R.T Tr. Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	225	61	13.5	2.6/C (3.75)	3	143	API Sect. 17	118 (130)	1920 (2209)
End-on	315	64.6	13.5	2.88/C (1.94)	3	143	API Sect. 17	335 (319)	2316 (2140)
Diag	270	68	13.5	3.86/C (3.36)	3	143	API Sect. 17	486	2877 (2727)

Table A.1.3: API 20th Edition 100 yr Environmental Criteria

Wav Dir	W.R.T Tr. Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	225	56.7	13	2.38/C (3.42)	3.5	135	API Sect. 17	106 (120)	1620 (1900)
End-on	315	59.85	13	2.63/C (1.77)	3.5	135	API Sect. 17	131 (125)	1796 (1670)
Diag	270	63	13	3.53/C (3.07)	3.5	135	API Sect. 17	118	2120 (2070)

The ultimate capacity values are not affected by the changes in environmental forces. However, the RSR's will change. The RSR based on the new environmental forces are tabulated below. The old values are given in ( ) for comparison.

Wave Dir.	W.R.T. Tr. North	20th Edition Base Shear	Ultimate Capacity	RSR
Broad Side	225	1620 (1900)	1861	1.15 (.98)
End-on	315	1796 (1670)	1964	1.09 (1.18)
Diag	270	2120 (2070)	1496**	0.71 (0.72)

**

In the diagonal direction, if the ultimate capacity has been determined based on excessive deck displacement (>4') rather than the first pile section fully plasticizing (see the report), it could have been increased to 1900 Kips. In this event, the RSR will be 0.90. It should be noted that the increase in ultimate capacity would not change the "fail" status of the platform as RSR is still less than 1.0.

PARTICIPANT D

REASONS FOR GROSS ERRORS

BENCHMARK ANALYSIS

1. Engineer's first use of both CAP and SACS (training).

2. Engineer's first use of API RP 20th edition methodology. The design wave heights for the 20th edition loading and the ultimate strength analysis loading were obtained by interpolating between the wave height factors given in Figure 2.3.4-4 for various approach angles. The correct method is to use the given wave factor for plus or minus 22.5 degrees.

3. Emphasis was on ultimate strength analysis, not design loading. Initial load pattern for ultimate strength analysis was generated by running a wave just under the deck and adding a lump sum deck loading to avoid generating wave loads on deck members. While this allows proper determination of ultimate strength, generating an acceptable load pattern, it under estimates kinematics on the jacket and deck. These values were mistakenly reported in the original submittal even though the correct values were used for deck loading. The corrected jacket loads now reported were calculated using the full wave height but with the deck members zeroed out for wave loads by using low C_d's and C_m's. The deck wave load was always calculated with the Section 17 method using the crest heights and velocities from the actual design wave.

4. Double checking of the reported results in the initial submittal failed due to the correct wave heights being reported when the shear values listed were for a smaller wave, not reaching the deck.

PARTICIPANT E

Subject: **Trials JIP - Benchmark Analysis**

Further to the Benchmark JIP meeting in October, we have prepared additional information for your inclusion in the final report. The API RP2A 20th edition wave loads (not in original report) are as follows:

Direction	Wave Approach Direction (from True North) (degrees)	RP2A, 20th Edition Metocean Parameters and Loads				
		Wave Ht. H−20 (ft.)	In−Line Current U−20 (ft/sec)	Wave Load on Jacket (kips)	Wave−in−Deck (kips)	Total Base Shear S−20 (kips)
Dir 1	225	56.7	1.56	2028	23	2051
Dir 2	270	63.0	3.37	2380	71	2451
Dir 3	315	59.9	3.14	2202	44	2246

Our original current were presented as total currents relative to platform coordinates, not in-line values as presented in the final report. The corresponding in-line values are:

Dir 1	1.70
Dir 2	3.67
Dir 3	3.39

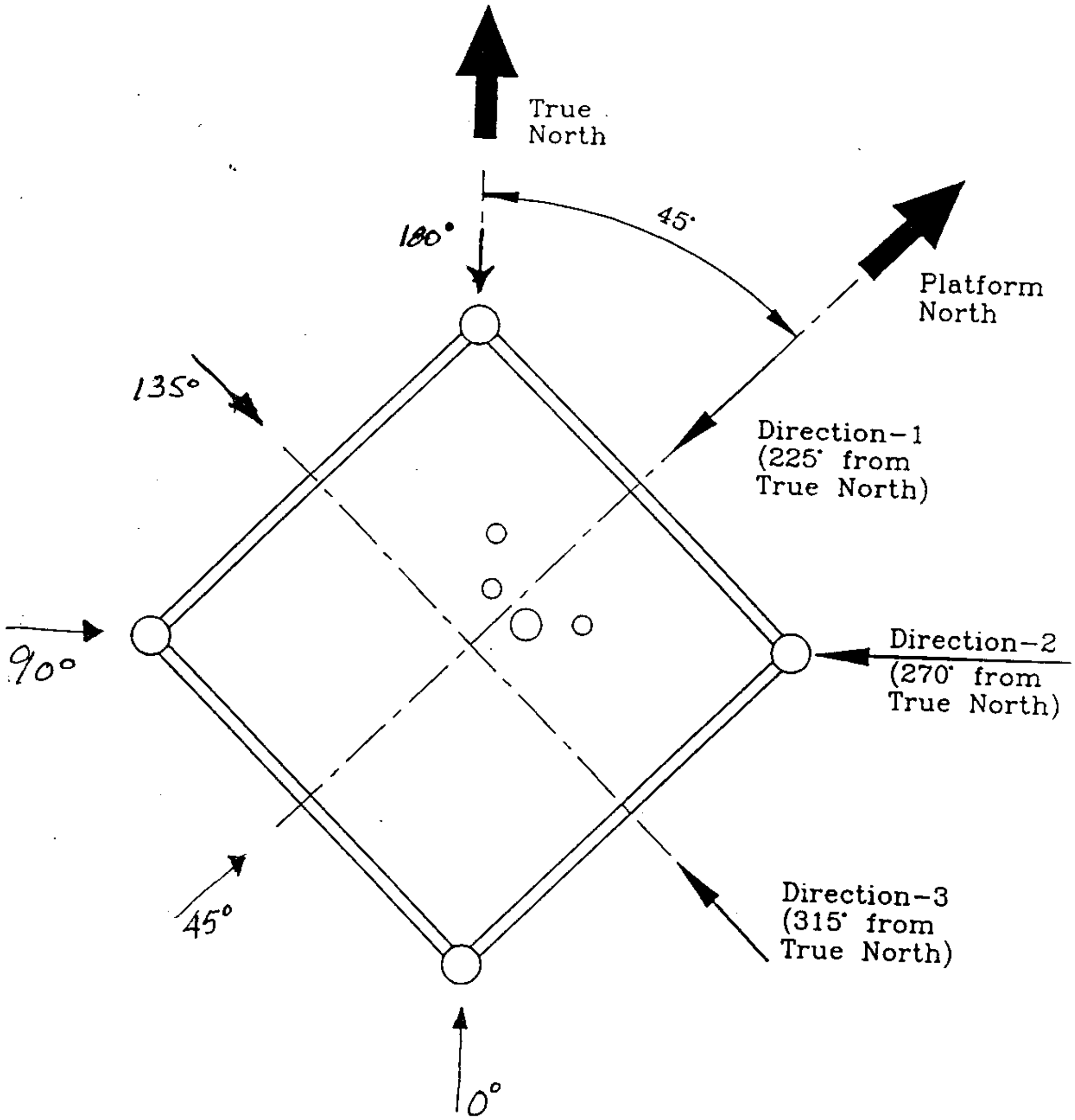
The RSR value for Direction 2 computed according to the enclosed 20th edition loads is 0.97.

PARTICIPANT F

The draft final report and the discussions at the final project meeting suggested that our submittal did not include any directionality effects in the analyses. You may recall that our analyses used the same wave height and current, etc., for both the 270 degree and 315 degree wave approach directions. Indeed directionality WAS considered and it was determined that the orientation of this particular platform justified using the full directionality factor of 1.00 for both the end-on/broadside and diagonal directions. The figure attached to this letter illustrates our logic.

The orientation of the platform of 45 degrees from true north puts the API RP-2A Figure 2.3.4-4 290° extreme wave approach direction almost directly between a "diagonal" and an "end-on/broadside" loading direction. The API RP-2A wave direction approach angles are specified to apply to bands ± 22.5 degrees. For this platform, a true diagonal direction would be 290° - 20° = 270° and a true end-on/broadside direction would be 290° + 25° = 315°. Considering the high degree of uncertainty in extreme wave approach direction (uncertainty certainly greater than 2.5°) and the degree of uncertainty in the survey of platform orientation, it seems prudent to apply the full environmental load to both the 270° and 315° wave approach analyses. It follows that if the full wave force is applied in the 315° analysis and the platform has symmetric framing that the 225° wave approach analysis (the other broadside/end-on direction) is not necessary. Again, the attached figure best illustrates my point and our logic.

We as engineers and designers need to recognize that uncertainties exist in nature and in the development of our design provisions. It is important, particularly in the assessment of existing structures where significant economic risk and significant threats to human safety and the environment are at stake, that we fully understand the assessment process and the details of the analyses we are performing. I feel that our interpretation of the directionality factors illustrates good engineering judgment and an appropriate application of RP-2A and its directionality provisions.



NOTE: The above three directions are basic directions referred to in the tables and figures. Tables 3-1 to 3-3 indicate normalized directions (with respect to True North) used in participants submittals.

FIG. 1 [redacted] **Wave Approach Directions**

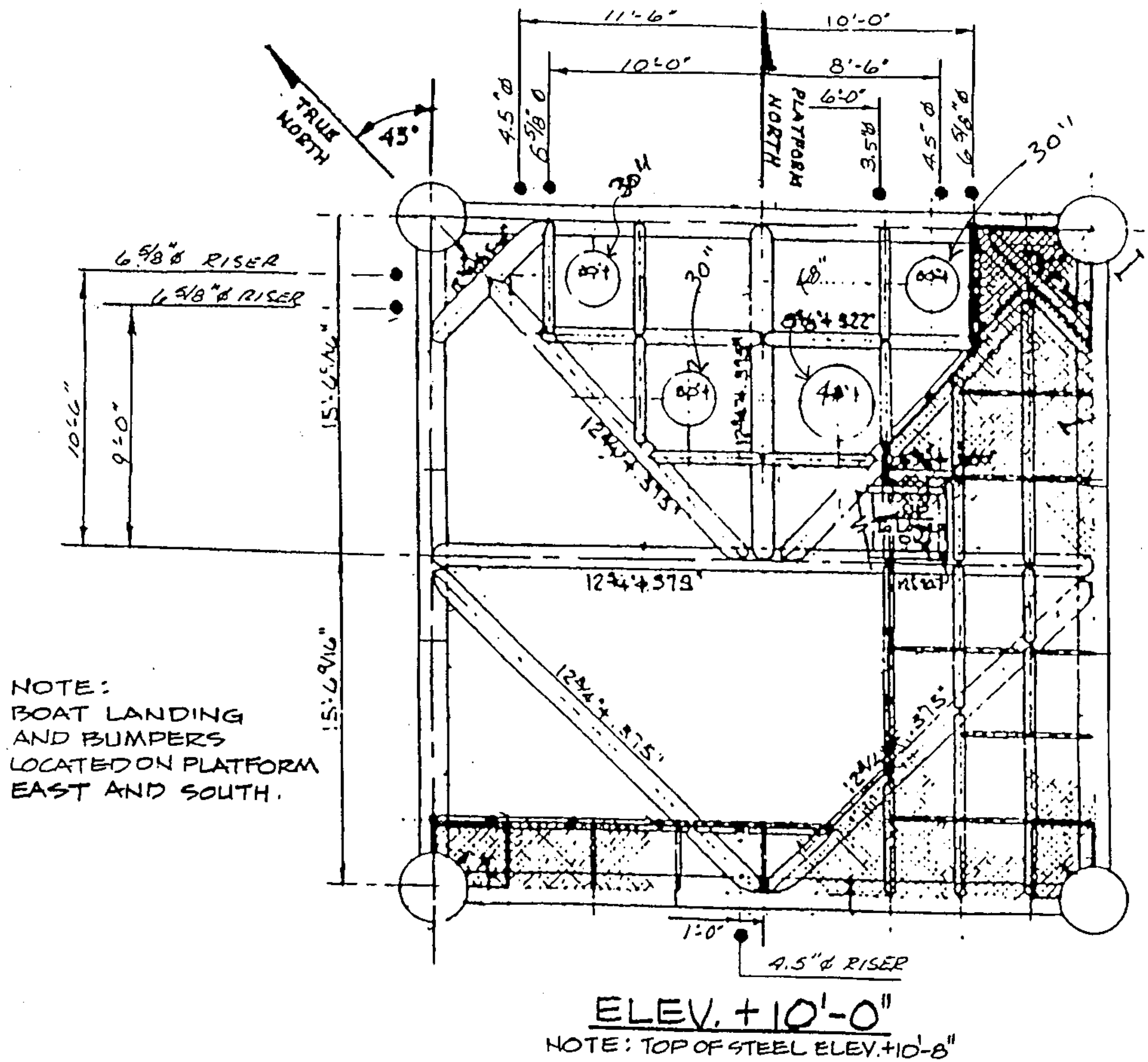
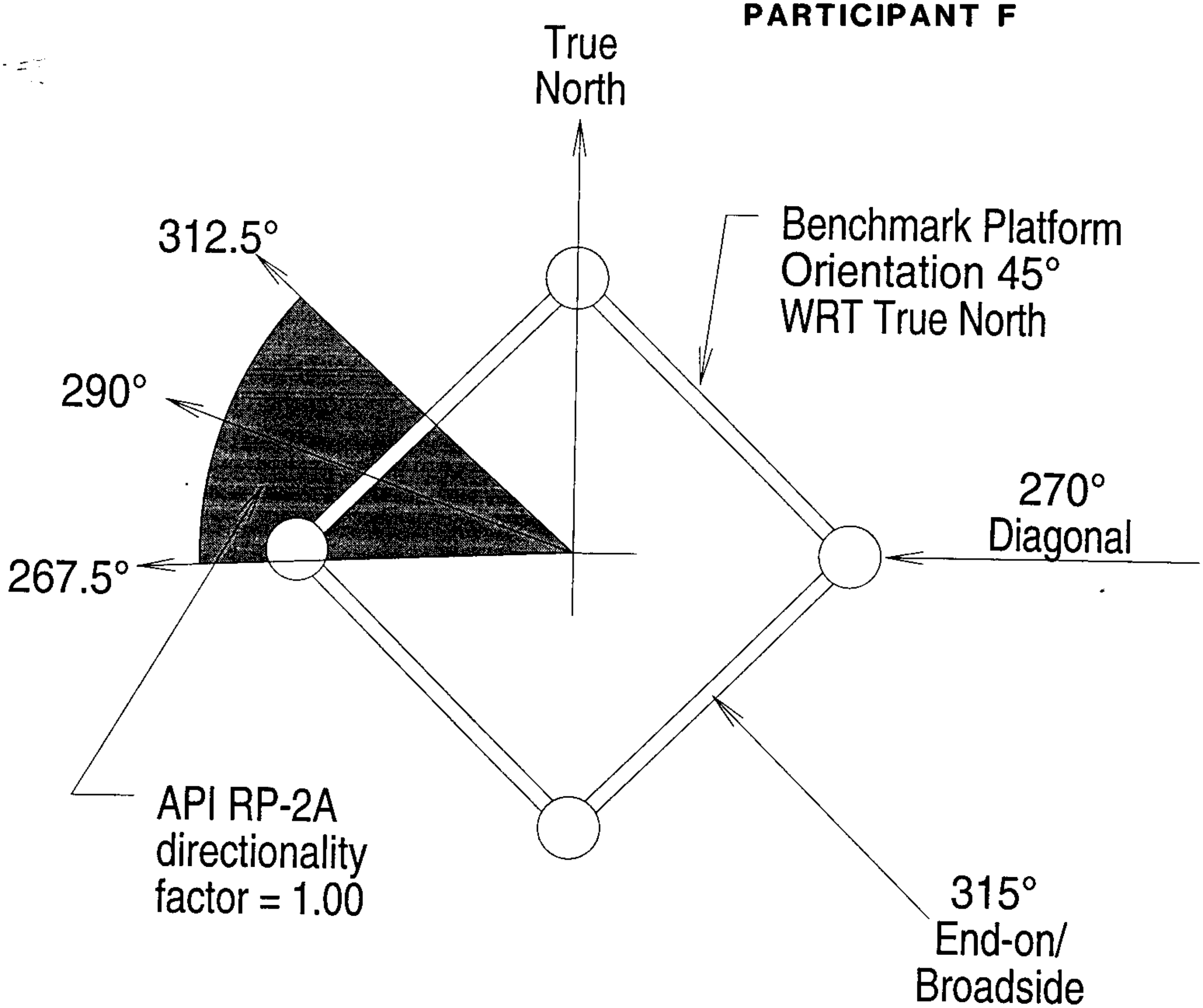


Figure 2[illegible] Key Plan - Benchmark Platform

PARTICIPANT F



The shaded region in the figure above illustrates the range of applicability of the full API RP-2A wave height (directionality factor = 1.00) to be 290° +/- 22.5°.

From the figure above, it is clearly apparent that the +/- 22.5 degree bands encompass both end-on and diagonal directions for this platform.
The 1.00 directionality factor was thus used in analyses for both the 270° and 315° directions.

PARTICIPANT G
ADDENDUM TO BENCHMARK DOCUMENT

This addendum provides additional comments on the benchmark analysis results for Participant G. These comments are a follow-up to the "Trials JIP - Benchmark Analysis" meeting conducted by PMB Engineering on October 19, 1994. This addendum addresses questions raised in the meeting regarding variations in the benchmark analysis results among the participants and the possibility of errors or oversights that may have been committed by some of the JIP participants.

Wave Loading

Participant G has reviewed its wave loading calculations on the benchmark platform and maintains that the input parameters used and the wave loading calculations are correct. This applies to the wave loading calculations for both the Section 17 Ultimate load (S-17) and the 20th Edition reference level load (S-20) documented for Participant G.

With regard to wave loading on the deck, Participant G explicitly modeled the tubular members at framing elevation +33.00 ft. and the tubular trussing between elevation +33.00 ft. and +42.13 ft. (here, member wave loads were calculated the same as that for the jacket members). Wave forces on the deck were considered only when the wave crest exceeded elevation +42.13 ft., which is the elevation of the cellar deck bottom of steel. Other JIP participants may have included the framing and trussing between elevations +33.00 ft and +42.13 ft as part of the silhouette area of the deck, which may explain why some participants calculated much larger wave forces on the deck.

Ultimate Capacity

The ultimate capacity of the benchmark platform determined by Participant G represents a lower bound of the results for all participants in the JIP. Two reasons can be cited to explain why the ultimate capacity results for Participant G are lower than the other participants:

Conductor modeling. Participant G conservatively modeled the well conductors only as wave load elements. It is likely that other participants also modeled the conductors as foundation elements, allowing the conductors to assist the piles in resisting wave shear on the platform. Participant G agrees that modeling the conductors as foundation elements is an acceptable practice, particularly for the benchmark platform (only four 36" OD piles with one 48" OD and three 30" OD conductors).

Soil modeling. The second contributor to the lower bound ultimate capacity by Participant G deals with soil modeling. Participant G used cyclic P-Y curves to define the soil lateral capacity, since it was considered that the cyclic criteria is more commonly used by other operators and design consultants. However, Participant G considers the cyclic criteria to be conservative in analysis of platform ultimate capacity and advocates to use of static P-Y curve formulations. It is likely that some other JIP participants did use static P-Y curves, contributing to higher calculated ultimate capacities.

If Participant G had included the well conductors in the foundation model and had used static P-Y curves for the soil lateral capacity, a much higher ultimate capacity would have been achieved for the benchmark platform.

PARTICIPANT I

PART B: REVIEW AND FEEDBACK TO API TG 92-5 (Addendum)

COMMENTS ON BENCHMARK DRAFT FINAL REPORT (PMB)

1. Wave Height vs Direction

Re. API RP 2A 20th Edition (p.26)
"Wave heights are defined for eight directions as shown in Figure 2.3.4-4. The factors should be applied to the omnidirectional wave height of Fig. 2.3.4-3 to obtain wave height by direction for a given water depth. The factors are asymmetry with respect to the principal direction, they apply for water depths greater than 40 ft., and to the given direction ± 22.5°. Regardless of how the platform is oriented, the 100-year omnidirectional wave height, in the principal wave direction, must be considered in at least one design load case."

Comment:
The Benchmark draft final report (PMB) showed that there is a variation of wave heights selected by the participants even though the metocean criteria are the same. This raised a question on how to interpret the guideline provided by API RP 2A 20th edition. For example, if the wave direction is happened to be in 222.5° from the true north (see Fig. 2.3.4-4), then what wave height factor should be applied? **0.90** or **0.75**? To be consistent, it is suggested that the interpolation of the wave height factors between two principal wave directions should be made if the wave direction is falling between them.

2. Reserve Strength Ratio, RSR = Ru / S-20

Comment:

The definition of the reserve strength ratio is clearly defined in API RP 2A Section 17.0 (draft). It is a good indicator to the strength of the platform in the normal engineering practice.

In review of BENCHMARK draft final report (PMB), it is noticed that the base shear (S-20) calculated by Participant "D" (see Tables 3-4,5 and 6) is much lower than the average value, consequently, the RSR values provided by Participant "D" are too high. The results of statistical data are somewhat skewed. This should be mentioned, at least, in the final report.

3. Follow-up on BENCHMARK Study Project

Comment:

The scope of work as proposed by PMB on BENCHMARK study has been carried out by PMB. Overall, PMB has done an excellent job on this project. Any additional work required to identify the cause of difference in the analysis results submitted by participants is beyond the current scope of work. It is suggested that some follow-up work should be initiated to resolve any outstanding problems, such as the explanation of why there is such a dispersion even in the base shear calculations etc. Is there any inconsistency of P-Y , T-Z and T-Q data generated by participants on BENCHMARK study project?

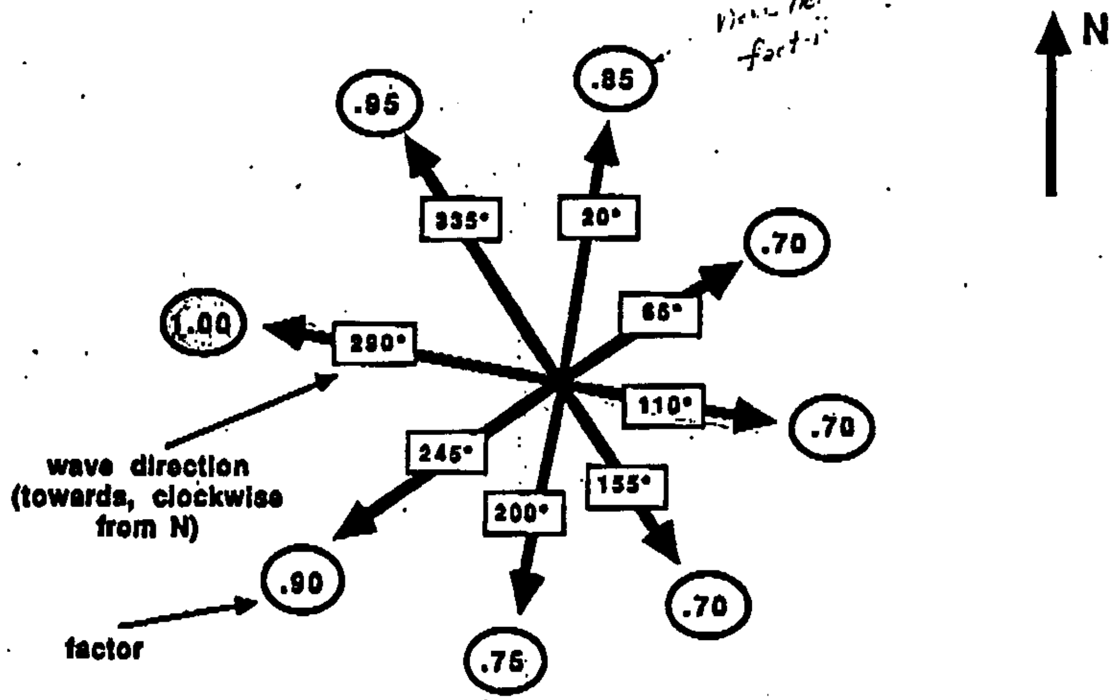
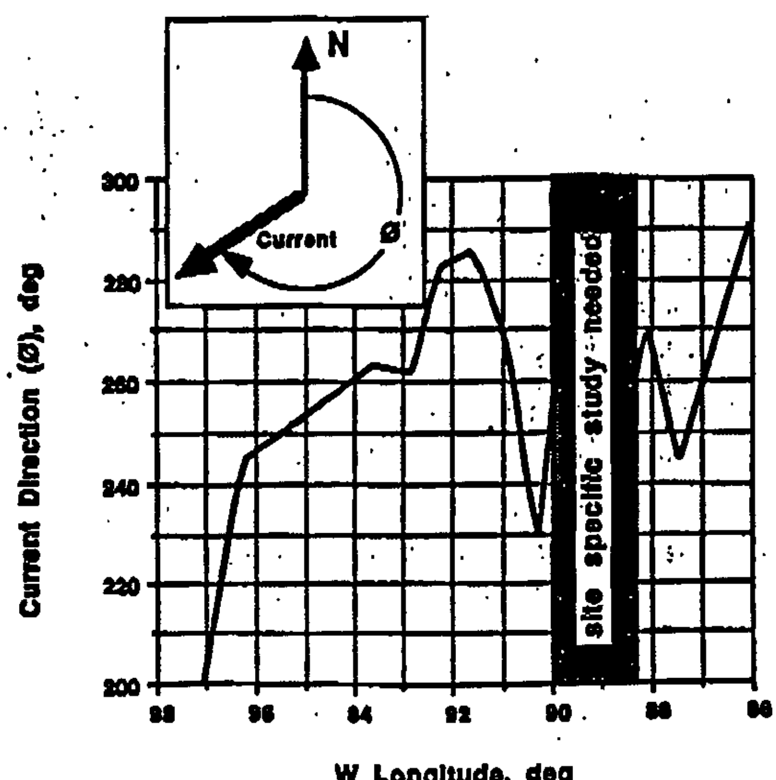


FIG. 2.8.4-4
GUIDELINE DESIGN WAVE DIRECTIONS AND FACTORS TO APPLY TO THE OMNIDIRECTIONAL WAVE HEIGHTS (FIG. 2.8.4-3), GULF OF MEXICO, NORTH OF 27° N AND WEST OF 86° W



W.D. < 150'

FIG. 2.8.4-5
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

PARTICIPANT I

ADDITIONAL INFORMATION ON BENCHMARK STUDY

1. Structural Modeling

The following simplifications were made in the structural model :

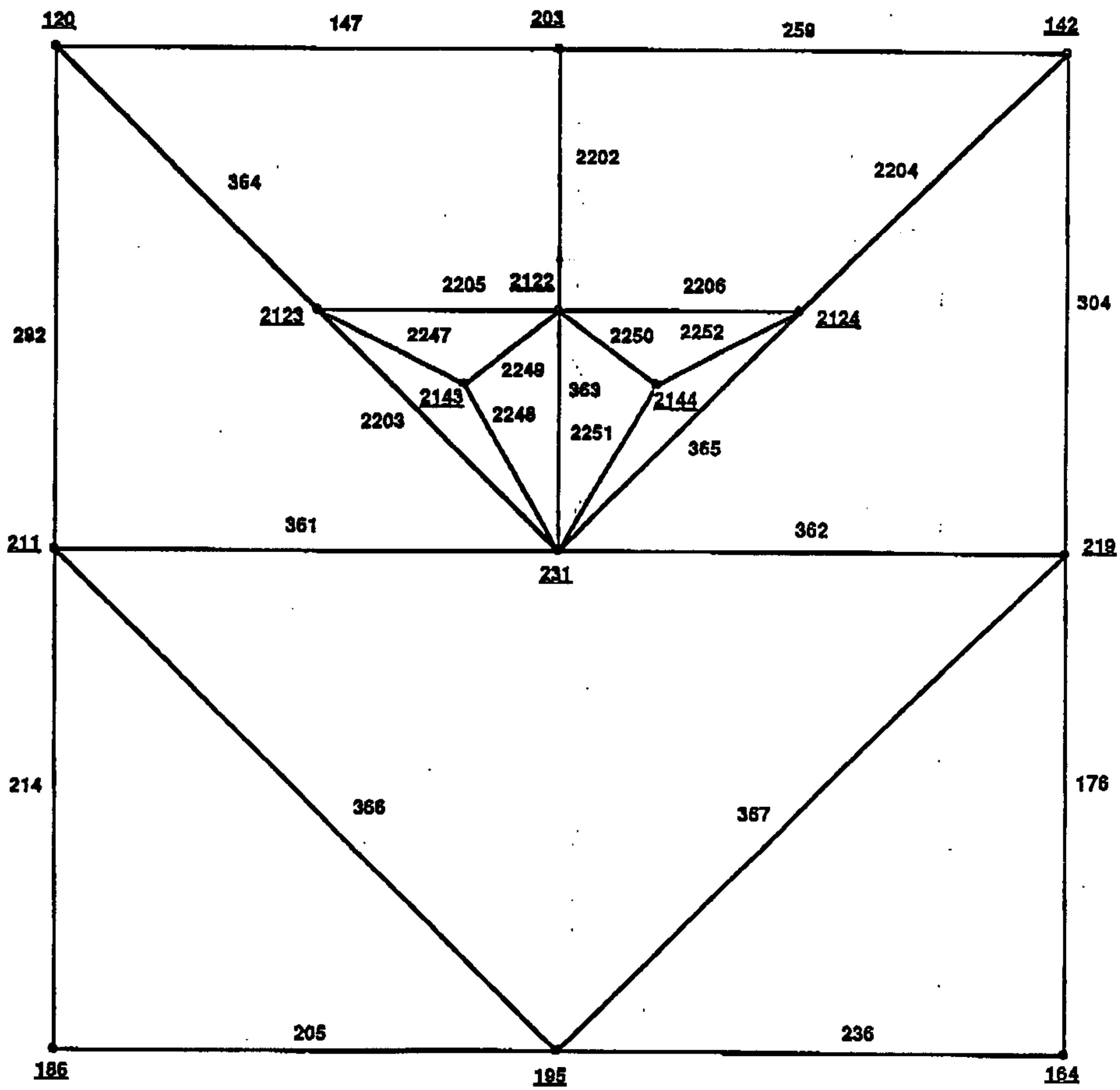
- Conductor Support at the Mudline - The conductor support is a hinge support at the mudline (Constraints in X,Y and Z directions - displacements only).
- Conductor Framing - The conductor framing is simplified. Did not use the actual framing as provided by PMB
- Conductor Modeling - Simplified. Use two equivalent conductors (Each conductor model consisting of two actual conductors).
- Boat Landing - Simplified. Use equivalent model, but not the actual framing configuration and member sizes provided by PMB.
- Wave Load on the Deck - For simplicity, the effects of wave load on the deck are neglected.

2. Load at 1st Member with Linear I.R. = 1.0 (Optional)

The lower S1 values reported by us are due to the fact that the 1st member with I.R. = 1.0 is located at the mudline, which is a horizontal diagonal member connected to the hinge support (conductor). Consequently, a portion of the lateral loads were transmitted to the hinge support through that particular member. This is a local effect caused by the structural modeling. If the conductor support is properly modeled (extended into the mudline), that particular member might have less value of I.R. See the attached sketch.

3. Wave Height vs Direction

Wave heights (H-17, H-20) of Directions 1, 2 and 3 as shown in Tables 3-1,2, and 3 of BENCHMARK draft report (PMB) were calculated using interpolation of the wave height factors between two principal wave directions. For example, for Direction 1 (225° from true North), the wave height factor is interpolated between two principal wave directions (245° and 200°), see FIG. 2.3.4-4 of API RP 2A 20th Edition.



BENCHMARK JACKET
EL (-) 157 ' - 0 "

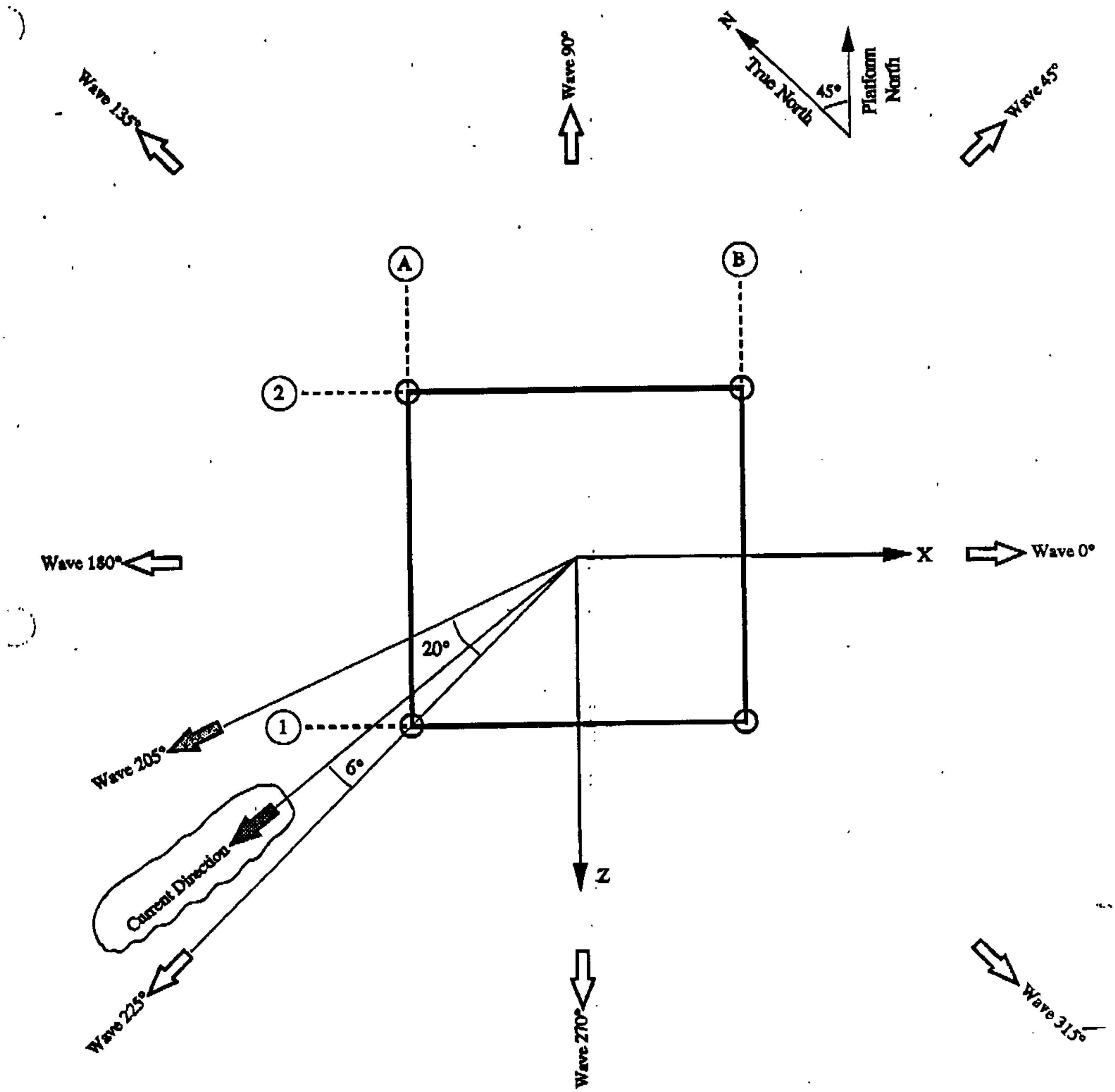


FIG. 1 **Platform Orientation & Wave Direction**

A.1 Data for Environmental Loads (Ultimate Strength Analysis):

Number of approach directions 4

1. Orientation with respect to Platform North 245 deg. (clockwise)

(205 deg.

Wave height	67.50 ft.
Wave Period	13.5 sec. (Tapp = 14.34 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stokes 5th Order Waves
Wave Crest	42.63 ft.

2. Orientation with respect to Platform North 270 deg. (clockwise)

(180 deg.

Wave height	65.61 ft.
Wave Period	13.5 sec. (Tapp = 14.18 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stokes 5th Order Waves
Wave Crest	41.14 ft.

3. Orientation with respect to Platform North 225 deg. (clockwise)

(225 deg.

Wave height	64.53 ft.
Wave Period	13.5 sec. (Tapp = 14.36 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stokes 5th Order Waves
Wave Crest	40.40 ft.

4. Orientation with respect to Platform North 180 deg. (clockwise)

(270 deg.

Wave height	56.23 ft.
Wave Period	13.5 sec. (Tapp = 14.05 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stokes 5th Order Waves
Wave Crest	34.21 ft.

PARTICIPANT J

1. We only carried out the "Section 17 Ultimate Strength" analysis and not the "API RP2A 20th Edition" analysis, hence all our results in the RP2A 20th Edition section of tables 3-1 to 3-3 should be blank. However, we notice that the Total Base Shear results from our Section 17 analysis have been incorrectly placed in the RP2A 20th Edition section of the results tables. This obviously, also applies to the first 2 columns of tables 3-4 to 3-6. Please could you see that this is corrected. For your aid we have included copies of the three tables indicating where the corrections should be made.

2. Also concerning the three tables described above, we were unable to supply the Wave load on the Jacket and the Wave-in-deck loading as these were not provided by the software which we used. We will, therefore have to leave those columns blank also.

3. In response to the specific questions raised at the meeting:

a) Wave-in-deck loading estimates, Wave crest elevation used?
   - No Wave-in-deck loading was calculated in our analysis due to restrictions in the software.

b) How the conductors were modelled? Were conductors modelled to contribute to foundation capacity?
   - Conductors were modelled as primary elements but were unable to carry any horizontal components of load. The conductors did not contribute to the foundation capacity.

c) Load level at first member with I.R. of 1.0. The members shall be identified. What increase in allowable stresses were considered in the computation of I.R.?
   - The load level of the first member with I.R. (assuming this is the Utility Ratio) of 1.0 in our case is the same as the load level of first member failure, already provided in our original report. This is due to the fact that in , the Utility Ratio is calculated based on the ultimate strength of the member taking into account the material properties and buckling considerations under combined loading. It is not

based on the allowable stresses specified in any codes with a percentage Safety Factor added on.

d) How the fixity was incorporated into the model?

- The model was taken to be fully fixed in all directions at the mud line level.

4. Finally, we would like to offer a brief explanation of why our ultimate strength analysis results were substantially higher than for other partners. We noticed from the results summaries, that most partners located collapse in the piles while we located it in the legs and braces. This was because our software doesn't yet model piles fully so we were using non-linear spring stiffnesses calculated from a separate software package. This meant that the failure of the springs could not be recognised in the analysis and so the jacket remained intact until the next members in the failure sequence reached their limit state, ie the legs and braces. This obviously means that we recognise yield at a later stage to the other benchmark participants.

PARTICIPANT K

Errors/Omissions in Analyses

1. We used a linear interpolation of the values in API RP 2A 20th Edn, Figure 2.3.4-4. rather than the prescribed +/- 22.5 degrees.

2. The centroid of the wind area was slightly offset and a small torque about the vertical platform axis was induced. Minor impact expected.

3. The wind load for the Ultimate Strength Analysis was based on the 20th Edition for new platforms rather than Section 17 for existing platforms. Very minor impact expected.

4. The wave blockage factor was assumed to be 0.845 in all directions, rather than the values ranging from 0.80 to 0.85.

5. The modelling of the conductor grid was simplified with two 'equivalent' members attached to be major horizontal framing. It appears that early failure of the lowest framing level for the required analyses submitted for the wave from platform East might be attribute to this modelling approximations. For the diagonal wave attack analyses, the bending stiffnesses were modified to improve the modelling, nevertheless local failure were still experience.

Continued....

Base Case - with Pile/soil effect considered:

	Direction 1 Deg fr TN 225	Direction 2 Deg fr TN 270	Direction 3 Deg fr TN 315
- Wave in deck loading estimates:	39.9kips	197.7kips	249.6kips
- Wave crest elevation used: (0ft @ mudline)	194.1 ft	202.6 ft	202.6 ft

- Conductor Modelling: Each conductor was individually modelled for hydrodynamic loading and stiffness above the mudline. The conductors were pinned laterally but released vertically at the guides. Below the mudline each conductor was modelled with static P-Y curves.

- Load Level at First Member IR of 1.0: This value was optional and was not computed. If computed it would have been based on the 20th Edition for new design with the 1/3 allowable increase and with minimum yield strengths.

Fixed Base Case - Without Pile/Soil Effect Considered

The jacket legs and piles were fixed some 12 ft below the mudline representing the depth of the inflection int he piles in a full analysis. All six degrees of freedom were fixed and a subsequent additional analysis with rotations released, demonstrated only marginal increase in flexibility.

PARTICIPANT M

After further investigating our benchmark analysis results for possible errors, we found that the base shear presented in our report does not account for the wave kinematic and current blockage factors. The revised base shear and RSR values are as follows:

Environment Direction	Base Shear in Report	Revised Base Shear	Revised RSR Foundation Case	Revised RSR Fixed Base Case
245°	2589.50 Kips	1992.8 Kips	1.29	1.79
290°	3265.18 Kips	2519.3 Kips	0.97	1.42
345°	2613.00 Kips	2027.6 Kips	1.29	1.73

The ultimate capacities remain the same.

Trials Joint Industry Project

Benchmark Analysis — Trial Application of API RP 2A — WSD Draft Section 17

Volume II — Participants' Submittals

by
PMB Engineering Inc.
San Francisco, CA

December 1994

PARTICIPANT "A"

1.0 SUMMARY

A static Push Over analysis was performed for a "Benchmark" platform located in the Gulf of Mexico. The soil information provided and the API RP 2A formulations were used to determine foundation response to the applied loads on the piles. The drawings provided and information made available on platform functional loads and member properties were used in the development of the computer model.

The computer model developed (see Section A.2.) was used in determining the applied environmental loads on the platform. Specific discussion of the benchmark analysis is presented in the following sections.

PART A: PLATFORM ASSESSMENT

A.1 ENVIRONMENTAL CRITERIA

Environmental criteria varied with storm analysis direction. The three directions chosen for the analysis were:

Platform Loading	Site Loading
North	Northwest
Northwest	North
West	Northeast

Wave, current and wind data for the platform site were determined based on both Draft Section 17 of API RP 2A and the 20th edition of API RP 2A. Directionality of current was accounted for in the combined wave and current to be acting on the platform.

The seastate criteria determined for this benchmark analysis are summarized as follows:

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
NORTH				Note 1:
Wave Height, H(ft)	45.0	50.9[1]	56.7	Includes 0.90
Wave Period, T (sec)	11.3	12.5	13.0	height correction
Current				factor
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.2	1.8	2.1	
Velocity (fps)	2.0	3.0	3.5	
Direction	North	North	270 deg	
Wave dir V (fps)	2.0	3.0	0.0	at -45 deg with
Computed Parameters				wave
V / gT	0.0044	0.006	0.000	
d / gT^2	0.038	0.032	0.000	
T_a / T	1.03	1.04	1.000	
Apparent Period, T_a	11.6	13.0	13.0	
H / gT_a^2	0.010	0.009	0.010	
d / gT_a^2	0.037	0.030	0.027	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	
WEST				Note 2:
Wave Height, H(ft)	45.0	53.1[2]	59.2	Includes 0.94
Wave Period, T (sec)	11.3	12.5	13.0	height correction
Current				factor
Blockage Factor	0.80	0.94	0.80	
Velocity (knots)	1.2	1.8	2.1	
Velocity (fps)	2.0	3.0	3.5	
Direction	West	West	270 deg	
Wave dir V (fps)	2.0	3.0	2.5	at 45 deg with
Computed Parameters				wave
V / gT	0.005	0.006	0.006	
d / gT^2	0.038	0.032	0.030	
T_a / T	1.03	1.04	1.045	
Apparent Period, T_a	11.6	13.0	13.6	
H / gT_a^2	0.010	0.009	0.010	
d / gT_a^2	0.037	0.030	0.027	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
NORTHWEST				Note 3:
Wave Height, H(ft)	45.0	54.8[3]	61.1	Includes 0.97
Wave Period, T (sec)	11.3	12.5	13.0	height correction
Current				factor
Blockage Factor	0.85	0.85	0.85	
Velocity (knots)	1.2	1.8	2.1	
Velocity (fps)	2.0	3.0	3.5	
Direction	NW	NW	215 deg	perpendicular to
Wave dir V (fps)	2.0	3.0	0.0	wave
Computed Parameters				
V / gT	0.005	0.006		
d / gT^2	0.038	0.032		
T_a / T	1.03	1.08	1.0	
Apparent Period, T_a	11.6	14.6	13.0	
H / gT_a^2	0.010	0.009	0.011	
d / gT_a^2	0.037	0.030	0.030	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	
NOTE: PLATFORM — TRUE NORTH — NE WEST — NW NORTHWEST — NORTH				

A.2 3-D MODEL GENERATION

A three-dimensional model of the platform was developed modeling all platform legs and vertical frame diagonal and horizontal plan braces. Member properties were as per the provided drawings. Deck plan framing was not specifically modeled, but represented as equivalent cross-braced framing to represent the stiffness and load transfer characteristics of the actual structure.

The generated computer model topology is shown on the following figure.A.3

A.3 SOFTWARE DESCRIPTION

The benchmark analysis was performed using I.D.E.A.S. developed computer program ASADS. ASADS is a three-dimensional linear and nonlinear static and dynamic analysis computer code with specific pre- and post-processor program enhancements for marine and offshore engineering applications.

Joint coordinates consist of a joint number or name and coordinates in a Cartesian (x-y-z) coordinate system. Beam-type member and other finite element incidences are defined to connect the joints as required. The model topology may be further refined by specifying joint and member releases and joint eccentricities at member ends.

Member properties amy be input directly or specified directly as a function of the member cross-sectional shape. Tubular, conical transition, AISC shapes, plate girder and box girder shapes are presently available and may be input as single segment prismatic or multiple-segmented variable sections.

The model may be further enhanced using ASADS' node mensuration and detailing facilities. The node mensuration facility determines joint chord and brace members, included angles between all intersecting members and prints warning messages relevant to the structure connectivity and sizing. This facility is particularly useful for checking the validity of the modelled structure and associated design.

Specified lead conditions consist of joint and/or member loads or joint displacements and/or rotations. Member loads may consist of concentrated, uniform or linearly varying member loads at or over any portion of the member.

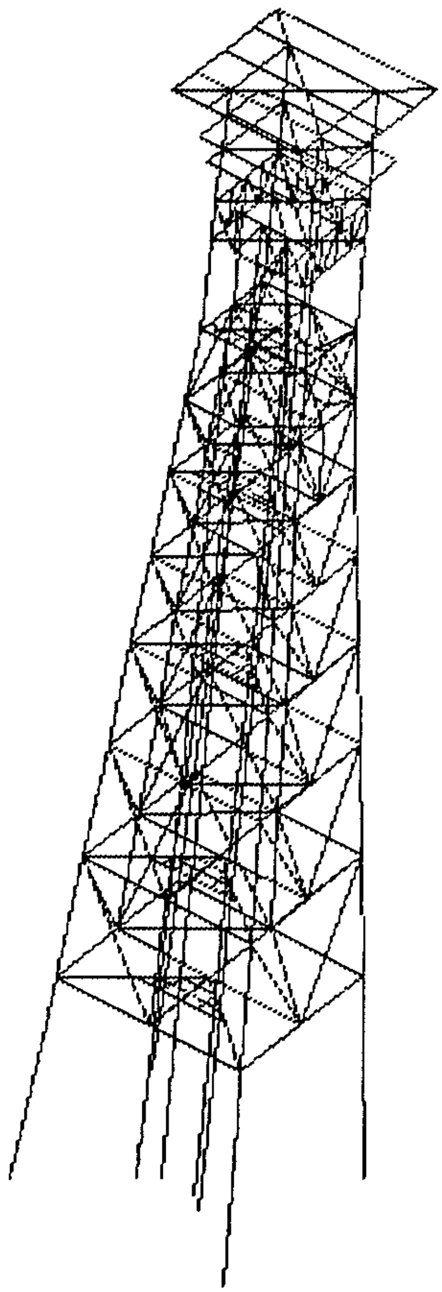
Automatic load generation is available for modelled dead, buoyancy, ballast, wind, wave and current loads. Direct dynamic inertial load generation for periodic wave and current loads is also available as a single-line command once a dynamic modal analysis has been performed.

ASADS' present analysis capabilities consist of:

- standard static linear stiffness analysis
- linear static stiffness analysis with non-linear pile-soil interaction
- iterative static "pushover" analysis
- dynamic modal analysis via subspace iteration
- dynamic spectral response analysis

Linear and nonlinear static stiffness analyses are performed via a one-line command with automatic bandwidth optimization included.

Iterative static pushover analyses were used for this benchmark analysis. This analysis capability is provided via a PC-ANSR module incorporated directly into ASADS. The program source code was



enhanced to allow processing of larger models typical of offshore platforms and to properly account for P-M interaction of tubular cross-sections as nonlinear beam-column elements.

This ASADS database is translated to a PC-ANSR compatible format along with additional user-specified parameters to describe the non-linear properties of the analysis model. ASADS offers the capability to directly transfer the nonlinear properties to the PC-ANSR module for previously specified tubular, wide flange, or plate or box girder members. Presently available nonlinear elements available include linear-eleastic truss, nonlinear truss, linear-eleastic beam-column, nonlinear beam-column, gap and nonlinear brace elements. PC-ANSR is made available to the public through the National Information Service for Earthquake Engineering - NISEE/Computer Applications, University of California, Berkeley.

A.4 SUMMARY OF APPLIED LOADS

Functional loads acting on the platform are as follows:

- Deck equipment and variables weight: 1,425 kips
- Deck and jacket selfweight (steel): 1,431 kips
- Jacket net buoyancy: 1,209 kips

The wind, wave and current loads generated along the three platform axes are as follows:

DESCRIPTION	NORTH	NORTHWEST	WEST
DESIGN LEVEL			
Base Shear (Kips)	806	775	777
O.M. (Kip-feet)	96,800	93,100	93,800
ULTIMATE STRENGTH			
Base Shear (Kips)	1244	1495	1331
O.M. (Kip-feet)	147,400	188,700	145,900
20TH EDITION API RP 2A			
Base Shear (Kips)	1033	1150	1474
O.M. (Kip-feet)	125,700	142,000	173,300

A.5 ULTIMATE STRENGTH ANALYSIS

An ASADS pushover analysis was performed for three storm wave directions; platform north, west and northwest. An ultimate strength storm load condition that maximized platform base shear was generated for each storm direction and incrementally applied to the model. The applied load increment was 5 percent of the ultimate strength level loading.

All tubular members were modeled as nonlinear beam-column members. Compression capacity was determined based on member slenderness ratios. The API recommendations for "K" were assumed, i.e., 1.0 for leg members and 0.8 for braces. The moment capacity of the member was reduced as a function of the present axial load per [Chen and Han, 1985], namely:

$$M / M_{cr} = \cos[(pi / 2) (P / P_{cr})]$$

The piles within the legs were modeled to provide the correct stiffness and the capacity of the components to resist axial and bending loads. Although the piles at jacket/pile interface have the geometric and material properties for strain hardening, this characteristic was not incorporated into this study.

Since all primary joints are leg joints and these are grouted to the piles within, all joints were assumed to be rigid and no joint failures were assumed in the analysis. Note that joint failures would have been considered if any of the leg joints were considered to be defective.

Tabulated and graphical results of these analyses follow. The tabulated results are presented on Tables A.5-1 through A.5-3 and the sketches facilitating the tracking process are presented in Appendix AA.

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	0.60	133	None		
4	1.29	266	None		
6	1.98	399	None		
8	2.67	533	None		
10	3.36	666	None		
12	4.05	799	None		
14	4.74	932	None		
16	5.43	1,065	None		
18	6.12	1,198	None		
20	6.81	1,331	None		
22	7.50	1,465	None		
24	8.20	1,598	140A	Single Hinge	Compr Pile
25		1,665	110A	Single Hinge	Compr Pile
26	9.23	1,731	2142	Buckling-SH	DH Step 28
27		1,798	2141	Buckling-SH	DH Step 29
28	10.47	1,864	881	Single Hinge	DH Step 32
29		1,931	255C	Single Hinge	DH Step 30
30	11.79	1,997	256C	Single Hinge	DH Step 31
			886	Single Hinge	DH Step 40
32	13.12	2,130	None		
33		2,197	2132	Buckling-SH	DH Step 37
34	14.47	2,263	None		
35		2,330	2111	Buckling-SH	DH Step 39
36	15.83	2,397	None		
37		2,464	292	Single Hinge	
			2655	Single Hinge	
38	17.27	2,530	130A	Single Hinge	Tension Pile
			456C	Single Hinge	DH Step 41
39		2,597	140	Buckling-SH	
			140P	Buckling-SH	
			287	Single Hinge	
			315	Buckling-DH	
40	19.05	2,663	120A	Single Hinge	Tension Pile
			215	Buckling-SH	DH Step 42
			256D	Single Hinge	
			415	Buckling-SH	DH Step 41
			655B	Single Hinge	DH Step 42
			655C	Tension-SH	
			656B	Single Hinge	
			2656	Single Hinge	
41		2,730	110	Buckling-SH	
			681	Single Hinge	
42	21.69	2,796	Numerous		
43	Collapse	2,796 +/-			

SH = Single Hinge, DH = Double Hinge

Table A.5-1
Ultimate Strength Analysis Results - Platform West Direction

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	0.66	150	None		
4	1.50	299	None		
6	2.35	449	None		
8	3.19	598	None		
10	4.04	748	None		
12	4.88	897	None		
14	5.72	1,047	None		
16	6.76	1,196	140A	Single Hinge	Compr Pile
17		1,271	886	Single Hinge	DH Step 27
18	8.03	1,346	881	Single Hinge	DH Step 22
19		1,420	255C	Single Hinge	DH Step 20
20	9.38	1,495	2132	Buckling-SH	DH Step 23
21		1,570	2141	Buckling-SH	DH Step 23
22	10.74	1,645	None		
24	12.93	1,794	120A	Double Hinge	Pile
26	15.40	1,944	None		
27		2,019	140	Single Hinge	
			2655	Single Hinge	
28	18.08	2,094	None		
29		2,168	140P	Single Hinge	
			287	Single Hinge	DH Step 30
			455C	Double Hinge	
			655B	Double Hinge	
			855C	Double Hinge	
			2112	Tension-SH	DH Step 30
			2121	Tension-SH	DH Step 30
30	21.25	2,243	255D	Single Hinge	DH Step 34
31		2,368	681	Single Hinge	
			858B	Single Hinge	DH Step 34
			2855	Double Hinge	
32	24.45	2,393	2656	Single Hinge	
33		2,468	487	Single Hinge	
			656B	Single Hinge	
			687	Single Hinge	
34	44.53	2,542	Numerous		
	Collapse	2,542 +/-			Compression Piles

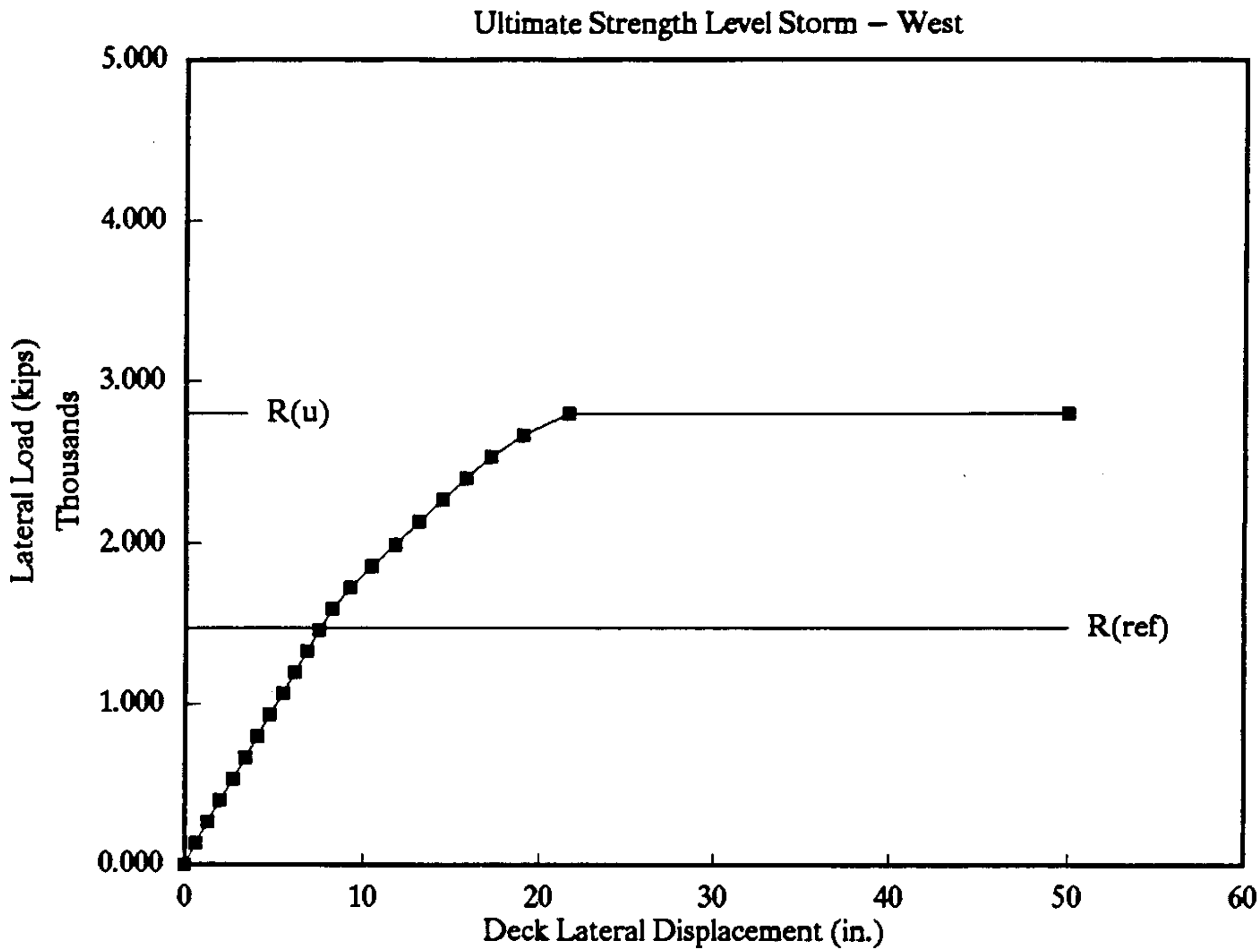
SH = Single Hinge, DH = Double Hinge

Table A.5-2
Ultimate Strength Analysis Results - Platform Northwest Direction

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	0.49	124	None		
4	1.15	249	None		
6	1.82	373	None		
8	2.48	497	None		
10	3.14	622	None		
12	3.80	746	None		
14	4.46	871	None		
16	5.12	995	None		
18	5.79	1,119	886	Single Hinge	DH Step 29
20	6.45	1,244			
21		1,306	881	Single Hinge	DH Step 31
22	7.11	1,368			
24	7.77	1,492			
26	8.43	1,617	130A	Buckling-SH	Compr Pile
27		1,679	140A	Buckling-SH	Compr Pile
28	9.30	1,741			
30	10.31	1,865	2131	Buckling-SH	DH Step 33
			2132	Buckling-SH	DH Step 33
31		1,927	256C	Single Hinge	DH Step 32
32	11.51	1,990			
34	12.73	2,114	255C	Double Hinge	
35		2,176	2655	Single Hinge	DH Step 42
			2656	Single Hinge	DH Step 42
36	13.95	2,238			
37		2,300	655B	Single Hinge	
			656B	Single Hinge	
38	15.19	2,363			
39		2,425	2141	Buckling-SH	DH Step 42
40	16.44	2,487	2122	Buckling-SH	DH Step 42
41		2,549	110A	Tension-SH	Tension Pile
			120A	Tension-SH	Tension Pile
			859B	Single Hinge	
42	17.85	2,612	Numerous		
	Collapse	2,612			

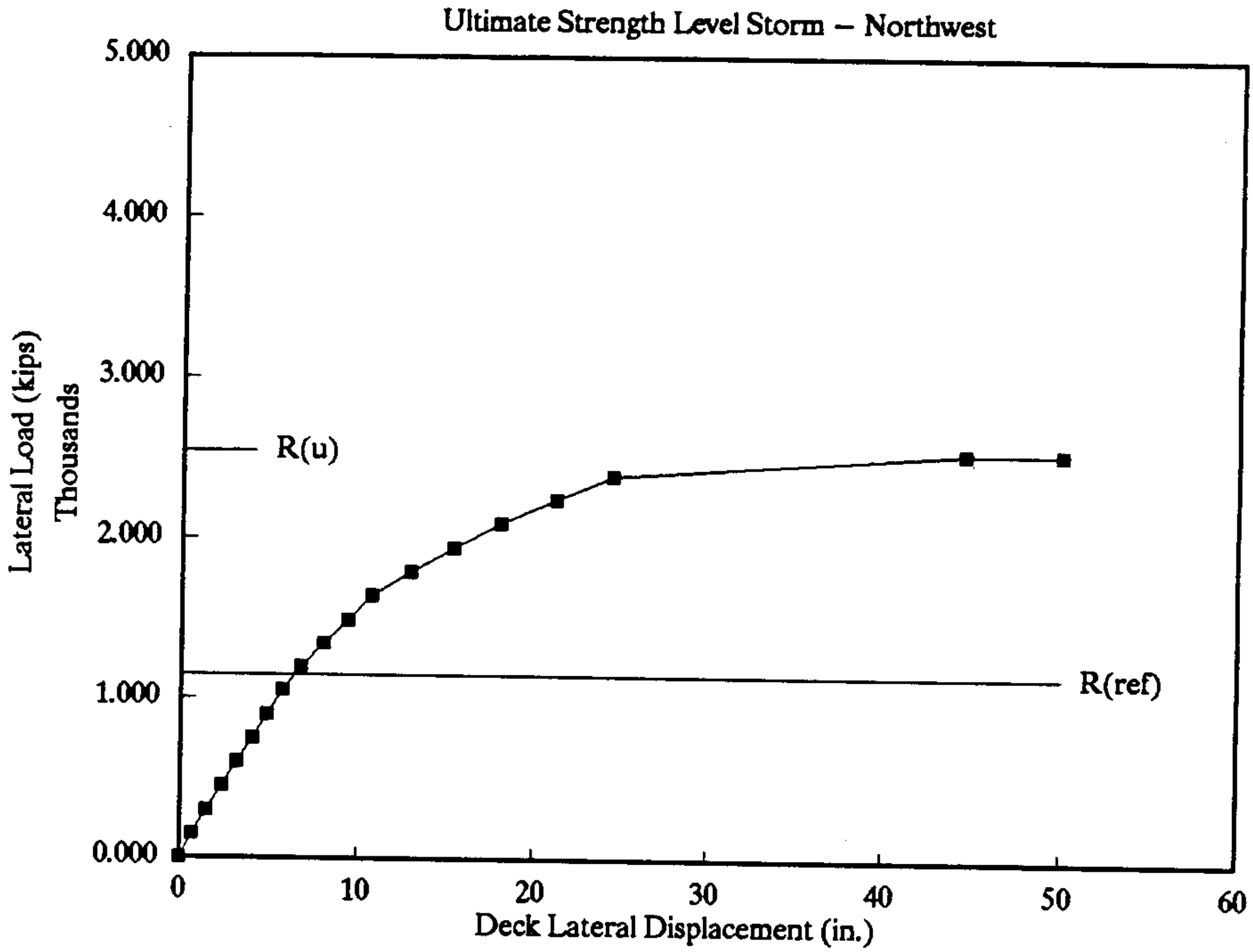
SH = Single Hinge, DH = Double Hinge

Table A.5-3
Ultimate Strength Analysis Results - Platform North Direction



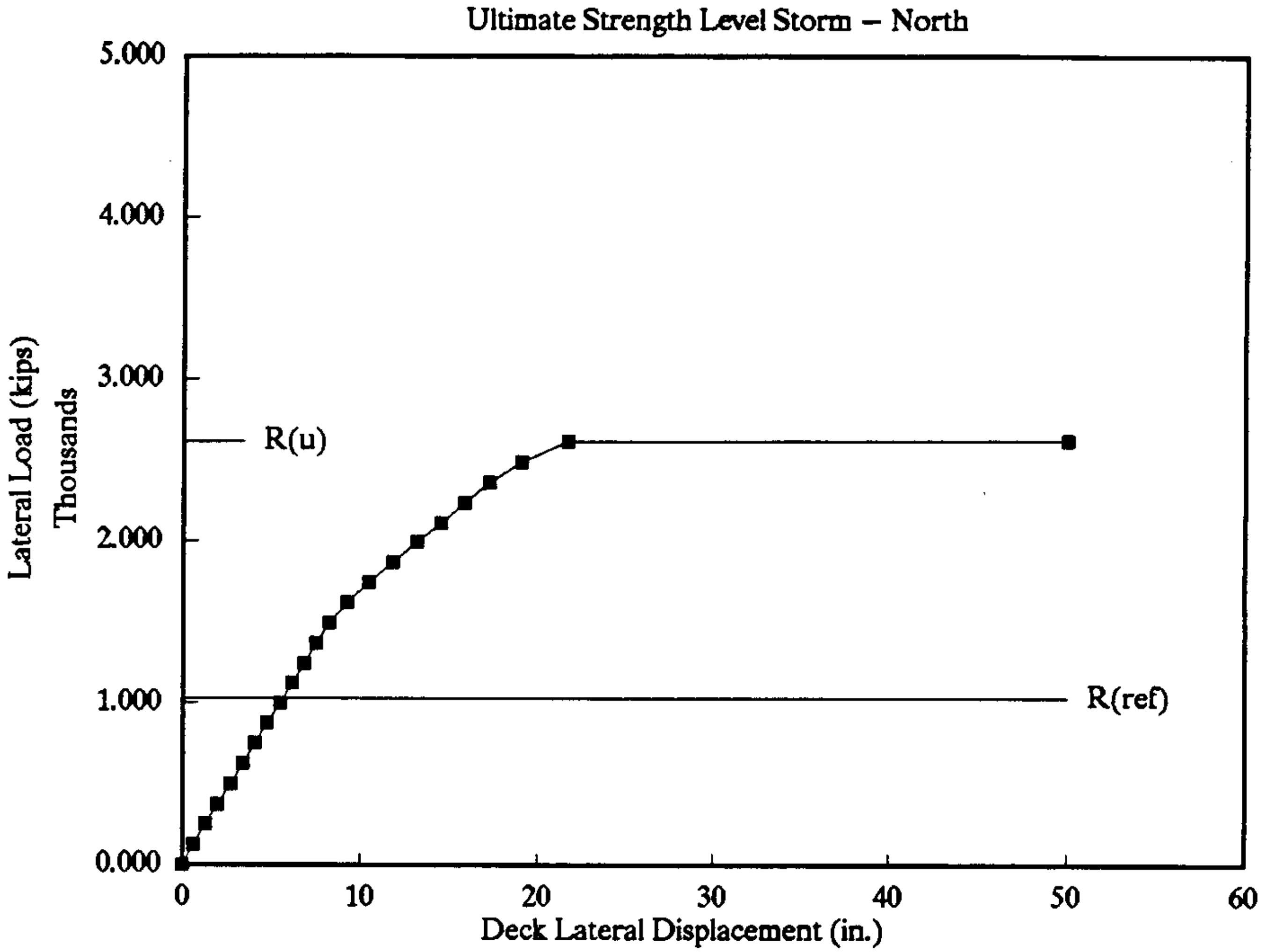
Reference Level Load, (S_{ref})	1,482 kips
Design Level Load (DLL)	777 kips
Ultimate Strength Level Load (USL)	1,331 kips
Ultimate Capacity (R_u)	2,796 kips
Reserve Strength Ratio (RSR) - to DLL	3.60
Platform Failure Mode: Jacket, Pile, Soils, etc.	Pile Compression

Figure A.5-1
Ultimate Strength Level Load-Displacement Results
Platform West Direction



Reference Level Load, (S_{ref})	1,150 kips
Design Level Load (DLL)	774 kips
Ultimate Strength Level Load (USL)	1,495 kips
Ultimate Capacity (R_u)	2,542 kips
Reserve Strength Ratio (RSR) - to DLL	3.28
Platform Failure Mode: Jacket, Pile, Soils, etc.	Compression Piles

Figure A.5-2
Ultimate Strength Level Load-Displacement Results
Platform Northwest Direction



Reference Level Load, (S_{ref})	1,056 kips
Design Level Load (DLL)	806 kips
Ultimate Strength Level Load (USL)	1,243 kips
Ultimate Capacity (R_u)	2,612 kips
Reserve Strength Ratio (RSR) - to DLL	3.24

Platform Failure Mode: Jacket, Pile, Soils, etc.

Figure A.5-3
Ultimate Strength Level Load-Displacement Results
Platform North Direction

PARTICIPANT "B"

PART A: Benchmark Analysis

A.1: Environmental Criteria

Analyses were performed in three wave directions aligned to platform broadside, endon and diagonal directions. The environmental criteria used to generate pushover load patterns in the these wave directions are given in Table A.1.1. API Section 17 environmental criteria for ultimate strength analyses are given in table A.1.2 for comparison. Table A.1.3 illustrates the 20th edition 100 year storm criteria. The pushover load pattern generated was incrementally applied on to the structure either until collapse due to significant nonlinear events or program termination due to analysis parameters reaching specified limits.

Table A.1.1: Environmental Criteria Used for Analyses

Wav Dir	W.R.T Plt Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	270	75	13.5	3.747/C	3	143	API Sect. 17	827	3692
End-on	180	75	13.5	1.941/C	3	143	API Sect. 17	653	2980
Diag	225	75	13.5	3.359/C	3	143	API Sect. 17	557	3168

Table A.1.2: Section 17 Ultimate Strength Environmental Criteria

Wav Dir	W.R.T Plt Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	270	61	13.5	3.747/C	3	143	API Sect. 17	130	2209
End-on	180	64.6	13.5	1.941/C	3	143	API Sect. 17	319	2140
Diag	225	68	13.5	3.359/C	3	143	API Sect. 17	486	2727

Table A.1.3: API 20th Edition 100 yr Environmental Criteria

Wav Dir	W.R.T Plt Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	270	56.7	13	3.424/C	3.5	135	API Sect. 17	120	1900
End-on	180	59.85	13	1.772/C	3.5	135	API Sect. 17	125	1670
Diag	225	63	13	3.069/C	3.5	135	API Sect. 17	118	2070

In addition to the criteria given in Table A.1.1, a wave kinematics factor of 0.88 was used in all wave directions. Current blockage factors of 0.8, 0.8, 0.85 were also used respectively in broadside, endon, and diagonal directions. Marine growth was assumed to be 1.5 feet constant to 150 feet below water line. Drag and inertia coefficients were selected according to API 20th edition recommendation. Wind forces were computed using the projected wind areas and the wind speeds given in Table A.1.1 as recommended by API 20th edition. Wave encounter and current stretching were not considered in the final analyses since they were found to have insignificant effect on the applied base shear in preliminary analyses.

The wave directions chosen in analyses correspond to the largest environmental forces on the platform with respect to platform primary axes. Due to this reason and due to platform symmetry, ultimate capacity of the platform determined in these directions will adequately represent the overall critical strength of the platform. Therefore, it was unnecessary to perform analyses in eight primary directions as suggested by API RP 2A Section 17.

A.2: 3-D Model Generation

The 3-D model of the structure was generated using the 1-D elements available in PMB's Capacity Analysis Program (CAP). The type of elements assigned to each component of the jacket are as follows:

Vertical Braces:	Marshall Struts	Inelastic Buckling
Horizontal Braces:	Beam-Column/	Lumped Plasticity Material Model
	Marshall Struts	Inelastic Buckling
Legs/Piles:	Beam-Column	Lumped Plasticity Material Model
Cond. Framing:	Linear Beam-Column	
Boat Landings:	Wave Load	No Stiffness
Deck Area:	Wave Load	No Stiffness
Deck Members:	Linear Beam	No Wave Loading
Soil:	PSAS	Inelastic as per API

The material yield strength was assumed to be 42 ksi. Effective length factor (K) for all vertical braces was assumed to be 0.65. For all other members, K factor was chosen according to API RP 2A 20th edition recommendations. The capacity of the jacket components were estimated based on the API RP2A-LRFD equations with the load and resistance factors set to 1.0. Equations in Section D.2.2.1, and Section D.2.3 respectively, were used to estimate the buckling and bending capacity of the members. The joint capacity was estimated based on procedures and equation outlined in Section

E.3.1.1 by assuming that the parameter Q_f equals 1.0. Nevertheless, it should be noted that the joint capacities based on API equations have been shown to be conservative. However, without sufficient experimental results to backup the physical observations presently available, it is difficult to consistently predict joint capacities contrary to that result from API equations.

The joint cans on the leg members were not explicitly modeled. The thickness for the entire length of the members was assumed to be the same as that at the midsection. The beam-column elements used to model the leg members assumes lumped plasticity model that allows plastic hinges to form at the end nodes only. Therefore, in the event of failure modes initiated by legs yielding, using these elements to model the legs without considering joint can thickness can cause premature yielding of those members. Premature yielding of leg members could result in erroneous nonlinear behavior of the structure. However, neglecting joint can thickness will not significantly affect the structural performance if its failure is controlled by modes other than the leg members yielding. To correctly model joint can thickness in CAP, one additional element at each end of leg members need to be introduced. This technique may substantially increase run time and possibly cause numerical difficulties. Since member failure modes of most K-braced structures are controlled by the braces buckling, neglecting leg joint cans in the member strength modeling is not expected to significantly affect the overall structural behavior. However, if the results indicate failure patterns on the contrary, this modeling assumption will need to be re-visited.

Modeling joint behavior has been a difficult task. Results from past analyses have shown that some of the techniques used gave questionable results (Andrew JIP, Phase 1). It has been proposed that joint modeling techniques should be studied carefully with some experimental backup. For these reasons, the joint behavior was not considered in the modeling. Nevertheless, two modeling techniques were considered in preliminary analyses. These techniques and the findings of the analyses are briefly discussed in Section A.5.

Only the primary members on the deck were explicitly modeled. Dead loads due to secondary members and equipment on each deck levels were applied as appropriate concentrated loads. All lateral forces on the deck due to wave were modeled by wave load elements representing equivalent areas computed based on the "silhouette approach" recommended in Section 17 of API RP 2A (provided by PMB). The drag and inertia coefficients for the deck area members were selected according to specifications provided by PMB. Boat landings were modeled as representative equivalent area wave load elements. Wave load elements do not provide any stiffness to the structure. Wind loads calculated based on API recommendations were applied as appropriate concentrated loads on deck-legs.

Piles were ungrouted. Legs and piles were connected by shim elements at each member end node. Shim elements have zero stiffness in the pile direction and provide high rigidity in the plane normal to the pile direction. This configuration forces the pile and casing nodes to move together in the lateral plane but slip freely along the pile direction. Conductors were modeled as linear beam elements with vertical and rotational restraints at the mudline. At intermediate conductor framing, deflections of conductor nodes were slaved in the global X and Y direction to appropriate nodes on the framing. Slaving deflections will provide a rigid link in the slaved directions between the master and the slaved nodes.

Soil was modeled by PSAS elements. PSAS elements represent the soil resistance by two orthogonal springs laterally and one spring axially. The lateral and the axial spring properties were based on the P-Y, T-Z and Q-Z curves generated based on API RP 2A 19th edition guidelines and static soil properties. The 20th edition changes affect only the T-Z curves generated in the inelastic regime. The automated feature that CAP has for this purpose is not capable of handling the 20th edition changes to the T-Z curve at the moment. Since the effects of the changes to the T-Z curves are insignificant on the overall ultimate capacity of the platform, using the 19th edition T-Z properties will suffice for the present purpose. It should be noted that for the diagonal direction wave considered, the lateral springs were rotated so that one spring is in the direction of the wave. Rotating the springs in the direction of the wave more realistically represent the soil resistance in the principle direction of lateral deformation.

A.3: Software Description

As mentioned earlier, the analyses were performed using PMB's CAP program. CAP essentially is a graphic interface to the finite element program SEASTAR. The graphical preprocessor provides users with an effective tool to build or modify a 3D structural model. The post processor allows users to check analysis results effectively and to present results with several graphical options.

The finite element library in SEASTAR program contains several inelastic and elastic type of elements. In a typical static pushover analysis, inelastic beam-column, Marshall strut, nonlinear truss and linear beam elements are used to model the jacket while PSAS and beam-column elements represent the nonlinear foundation. The deck members are usually modeled by linear beam elements. Wave induced deck forces and boat landings are modeled by wave load elements. The beam column elements in SEASTAR do not have inelastic buckling capability. The Marshall strut is capable of modeling inelastic buckling, however, it does not carry moments. All the inelastic elements in SEASTAR are capable of modeling fracture behavior.

The wave load is typically applied as a load pattern which is generated based on given wave and current profile. Typical load pattern generation is based on regular waves. The load pattern is applied incrementally as nodal loads until platform failure is reached.

CAP provides Preconfigured macros to generate pushover load patterns and to perform static pushover analyses, among others. In most cases, the default parameters set in these macros are adequate to perform the necessary analyses. Fine tuning these parameters may result in more efficient analysis runs. However, changing the analysis control parameters in the static pushover macros is recommended only for those with significant experience in performing nonlinear analyses.

The new static pushover solution algorithm available in SEASTAR is quite efficient compared to the pseudo static algorithm from the older versions. Nonetheless, with the default settings for the analysis parameters, the analysis runs averaged six to seven hours for each direction considered on a SUN SPARC10 machine. However, it should be noted that some of the analyses probably could have been terminated much before the default 300 load steps which would have resulted in significant reduction is run time.

A.4: Ultimate Strength Analysis Results

This section addresses the ultimate strength results that is required to be presented in the JIP report. Results from analyses performed by using a full nonlinear model are presented in Tables A.4.1, A.4.2 and A.4.3, respectively, for broadside, endon and diagonal wave directions. Lateral load/displacement curves are presented in Figures A.4.1 through A.4.3 for the respective wave directions. The lateral displacements plotted refer to the node (1219) at deck level +43 feet at the SE corner leg intersection. The lateral loads refer to the base shear induced by the pushover load generated based on the environmental conditions given in Section A.1. The base shear was computed in a horizontal plane at elevation -158 feet (1 feet below mudline). Figures A.4.4 through A.4.6 illustrate the component failure sequence and the deformed shape (magnified) of the structure at analysis termination.

A.5: Commentary on Ultimate Strength Analysis Results

The analyses results presented here indicate that in all three wave directions, the failure of the platform is primarily governed by the piles yielding and eventually plasticizing. In all three directions, the first components to reach yield were the leg members in the bottom bay. The piles connecting to the pile head nodes and the leg-pile members in the bottom leg members yielded following the leg members. The pile section at 90' below mudline where the wall thickness reduces from 1.875 inches to 1.5 inches fully plasticized for deck deformations in the order of 3 feet. All members that yielded were dominated by bending stresses, as expected. No other components in the jacket reached their yield or buckling capacities. The largest axial forces were in the bottom bay vertical braces which were over 100 kips less than their buckling capacity.

The type of failure observed in the analyses indicate extremely weak foundation strength. This behavior is not entirely unexpected since the top 100 feet of soil layer is soft clay with a shear strength profile varying from 80 pcf at 0 feet to 600 pcf at 100 feet. Failure was most likely initiated by the large lateral deformation of the pile sections near mudline due to insufficient lateral resistance provided by the weak top layer of soil. As listed in Section A.2, the legs and piles were modeled by using beam-column elements which have strain hardening properties in the nonlinear regime. Since no braces buckled in the structure, the nonlinear behavior of the structure was entirely governed by the yielded legs and piles. Due to the strain hardening behavior of the yielded legs, there was no reduction in lateral load capacity of the structure in the nonlinear regime. In this event, the analytical lateral load capacity of the structure increases until the analyses terminate due to numerical difficulty or complete collapse of the structure due to excessive deformation. Under these circumstances, a reasonable estimate of the ultimate capacity was made as the lateral load at which the first pile section fully plasticized.

As discussed in Section A.2, joint cans on the legs were not explicitly modeled. This modeling assumption is based on behavior of typical K-braced jackets where the ultimate capacity is governed by braces buckling. However, the results observed here was quite different from that originally anticipated. Therefore, an analysis with the leg thickness increased to account for the joint cans was performed in the broadside wave directions to verify the significance of this modeling difference. The results showed no significant change in the ultimate capacity in broadside direction. The mode of failure remained controlled by piles or legs yielding except that the first members to yield were the top pile sections instead of the bottom bay leg members as in the earlier analysis. It was inferred that the results in the other two wave directions would also show similar tendencies. Thus, the modeling assumption to neglect joint cans in the analyses is acceptable.

In an attempt to model joint behavior, joints that were weaker than the braces (strength less than buckling capacity) were intended to be modeled by using equivalent nonlinear truss members. These truss members have yield capacity equivalent to joint strength. Joints are modeled by replacing the braces with these equivalent truss elements. The post yield behavior of the joints may be modeled either as strain hardening or fracturing types. It should be noted, however, that the fracture modeling of joints is a complex issue and there is insufficient experimental data to back up any assumptions made in using this technique. One example of the complexity of this issue is the determination of the strain at which fracture initiates. The selection of the fracture strain essentially controls the prediction of the joint strength and thus will significantly affect the ultimate capacity of the jacket. Since proper experimental data are unavailable, the fracture strain will be assumed to be equal to the yield strain for any fracture type joint modeling considered here. Therefore, at reaching yield capacity, the equivalent joint members would be completely severed from the chord member. Thus, no brace support will be provided to the bays at which joint failures occur. This failure mode typically leads to a lower bound value of the ultimate capacity. Strain hardening approach on the other hand, may not allow correct redistribution of the forces in the structure after the joints yield. Since load shedding due to failure of joint is not captured by this type of modeling, the resulting ultimate capacity can be over estimated. Local buckling effects and joint flexibility were not considered in modeling.

Upon investigating K-joint capacities, it was found that they were stronger than the braces that frame into them. Under this circumstance the braces will have buckled before joints reached capacity. Therefore, it was not necessary to use either one of the techniques discussed above to model "weak joints" in any of the analyses.

In another analysis performed to verify the load path dependency of the structure, API RP 2A Section 17 ultimate strength environmental criteria were used in the broadside direction. The wave height of 61 feet in the latter analysis produced only nominal deck loads unlike the runs corresponding to the 75 feet wave in the earlier analysis. The pushover load generated was incrementally applied as in other analyses. The failure path and the ultimate capacity from this analysis had no significant changes from the earlier analysis.

Finally, analyses were performed to verify the effect of fixing the base of the platform. Results from these analyses in broadside and diagonal directions are presented respectively, in Figure A.5.1 and Figure A.5.2.

PART B: Review and Feedback to the API TG 92-5

B.1: Ultimate Strength Analysis Criteria- Environmental

In Figure 17.6.2-4, the caption should indicate that the directions and factors also apply to currents.

B.2: Structural Assessment

Guidelines to select suitable analysis method (linear global, local overload or global inelastic) given in Section 17.7.3a through 17.7.3c should be more clearly stated.

Item 3.b and 3.c in Section17.7.3c do not address the issue of modeling braces that carry significant moments. One example is braces that frame into pile heads.

Item 3.d in Section 17.7.3c does not clearly state what the actual loads or the loads based on the strength that act on joints. Some joint modeling techniques should be stated here with their advantages and disadvantages.

B.3: Commentary of Ultimate Capacity Evaluation

See Section A.5

Table A.4.1: Ultimate Strength Analysis Results for Broadside Wave (270° PN)

Lateral load level for first member with unity check =1.0# **847** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
76	2.005	1197	Leg Members A1-1, A2-18, B1-35, B2-52	Initial Yield	Bottom bay leg Members - First component to fail B/C element first yield @ node I
78	2.068	1221	Pile Members B2-455, B2-456	Initial Yield	SE and SW piles at 90' below mudline (ML) B/C elements first yield @ node J
80	2.252	1287	Pile Members B2-451,452,457,458,501, 502,509,510	Initial Yield	Pile sections at 90-100' below ML and at ML on all 4 piles B/C elements first yield @ node J
90	2.413	1334	Pile Members B2-451,455,456,462	Initial Yield	Pile section at 90-100' below ML B/C elements first yield @ nodes I&J
96	2.542	1355	Leg-Pile Members LpileA1-2, LpileA2-19	Initial Yield	Bottom bay leg pile members B/C element first yield @ node I
210	3.269	1776	Level 1 horizontals Lev1H-126,154	Initial Yield	Level 1 horizontal B/C element first yield @ node I
286	3.760	1861	Pile Member PileB2-453	Fully Plastic	NE corner pile at 90' below ML. Lateral load taken as ultimate capacity

Table A.4.2: Ultimate Strength Analysis Results for Endon Wave (180° PN)

Lateral load level for first member with unity check =1.0# **641** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
77	1.974	1186	Leg Members LegB1-35, LegB2-52	Initial Yield	Bottom bay leg members - First component to fail B/C elements initial yield @ node I
78	2.014	1203	Leg Member LegA1-1	Initial Yield	Bottom bay leg members B/C elements initial yield @ node I
79	2.045	1215	Leg, Pile members LegA2-18, PileB2-458	Initial Yield	Bottom bay leg member and pile section on NW comr pile at 90' below ML B/C elements initial yield @ node I
82	2.277	1299	Pile Members PileB2-457,462,503,512	Initial Yield	Pile sectins at 90-100' below ML and at ML B/C elements initial yield @ node J
92	2.509	1352	Leg Pile Member LpileB2-36	Initial Yield	Leg pile B2 in bottom bay B/C elements initial yield @ node I
114	2.519	1330	Pile Members PileB2-455,456	Initial Yield	Pile members at 100' below ML B/C elements initial yield @ nodes I&J
204	3.019	1603	Horizontal Member Lev1H-71	Initial Yield	Horizontal member at level 1 B/C elements initial yield @ node I
453	4.360	1949	Pile Member PileB2-454	Initial Yield	NW pile section 100' below ML B/C element initial yield @ nodes I&J
486	4.402	1964	Pile Member PileB2-454	Fully Plastic	NW pile section 100' below ML Lateral load taken to be ultimate capacity

Table A.4.3: Ultimate Strength Analysis Results for Diagonal Wave (225° PN)

Lateral load level for first member with unity check =1.0# **807** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
91	1.898	1166	Leg Member LegA1-1	Initial Yield	Bottom leg section on leg A1 - First component to fail B/C element first yield @ node I
92	1.996	1219	Leg and Pile Members LegB1-35, PileB2-501,509	Initial Yield	Bottom bay leg and piles at ML B/C elements first yield @ node I
93	2.055	1248	Leg Members LegA2-18, LegB2-52	Initial Yield	Bottom leg sections on legs A2, B2 B/C elements first yield @ node I
94	2.106	1271	Leg Pile Member LpileA1-2	Initial Yield	Bottom pile member in leg A1 B/C elements first yield @ node I
98	2.361	1376	Pile Members PileB2-456,503,512	Initial Yield	Pile section at ML and at 100' below ML B/C elements first yield @ node I or J
128	2.533	1397	Pile Member PileB2-456	Initial Yield	SE pile at 100' below ML B/C element first yield @ nodes I & J
131	2.545	1398	Pile Member PileB2-451	Initial Yield	SW pile at 100' below ML B/C element first yield @ nodes I & J
224	2.938	1496	Pile Member PileB2-456	Fully Plastic	SE pile at 100' below ML Lateral load taken as the ultimate capacity

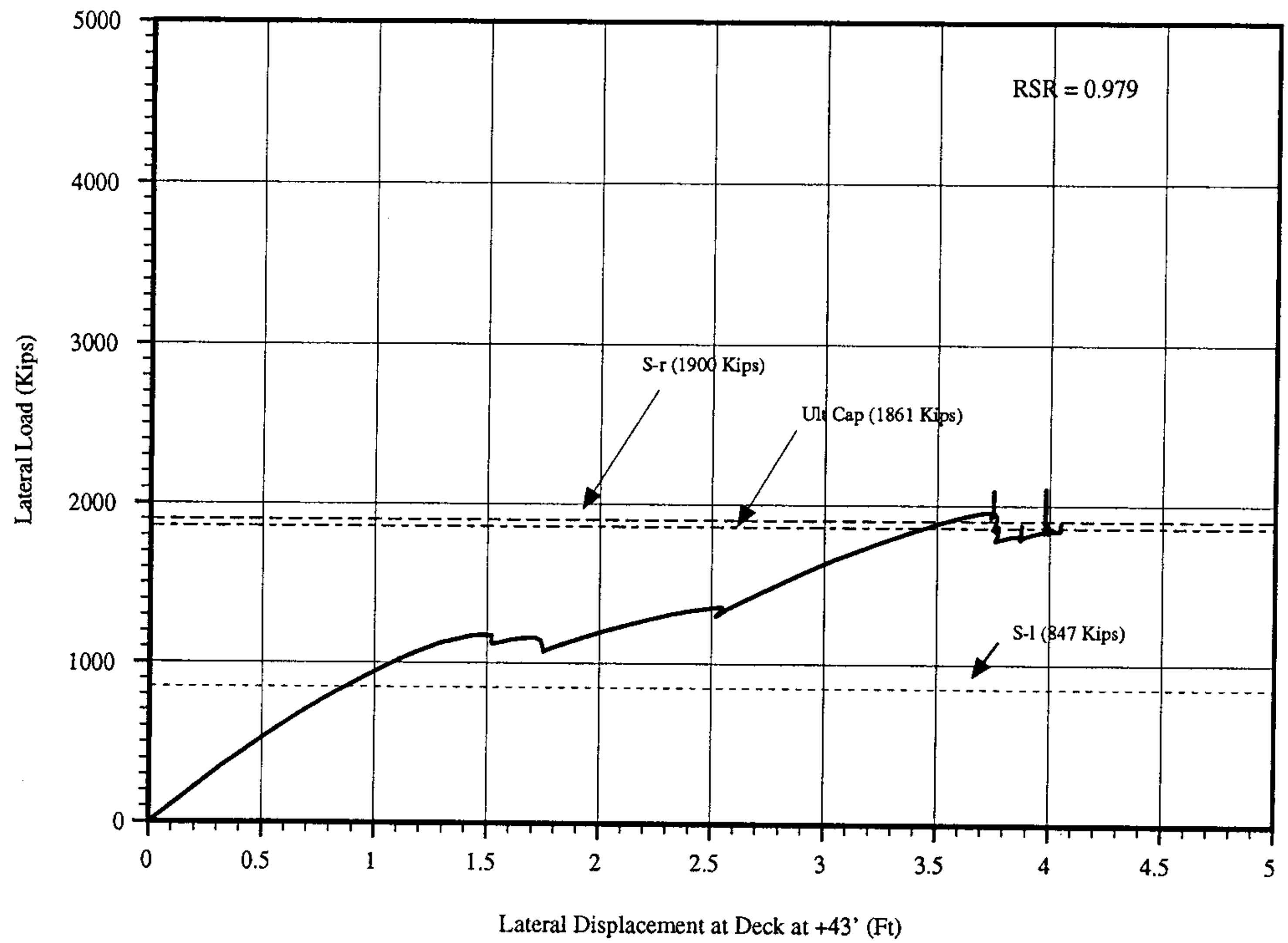


FIGURE A.4.1: Pushover Analysis Results - Broadside Wave (270° PN)

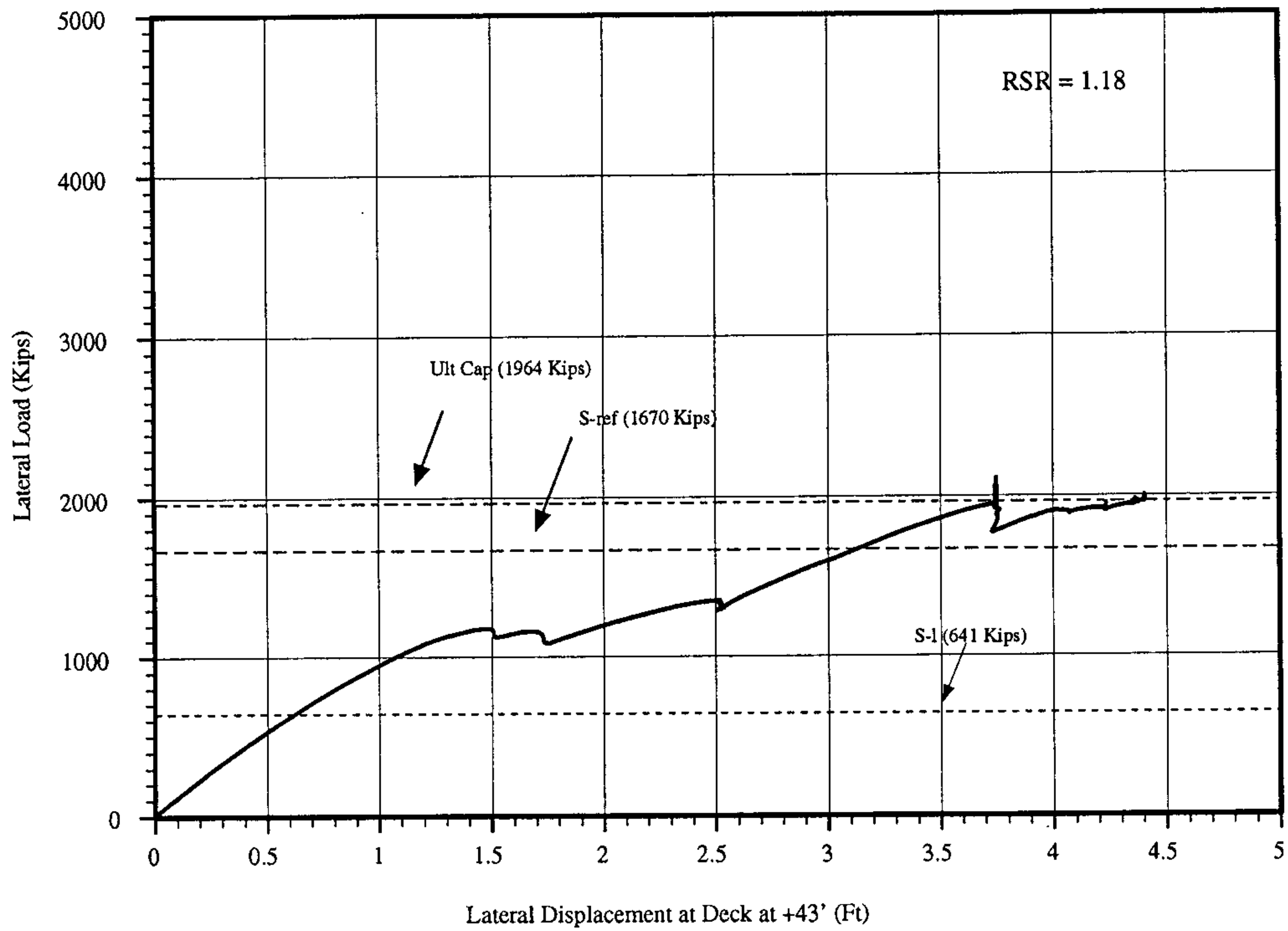


FIGURE A.4.2: Pushover Analysis Results - Endon Wave (180° PN)

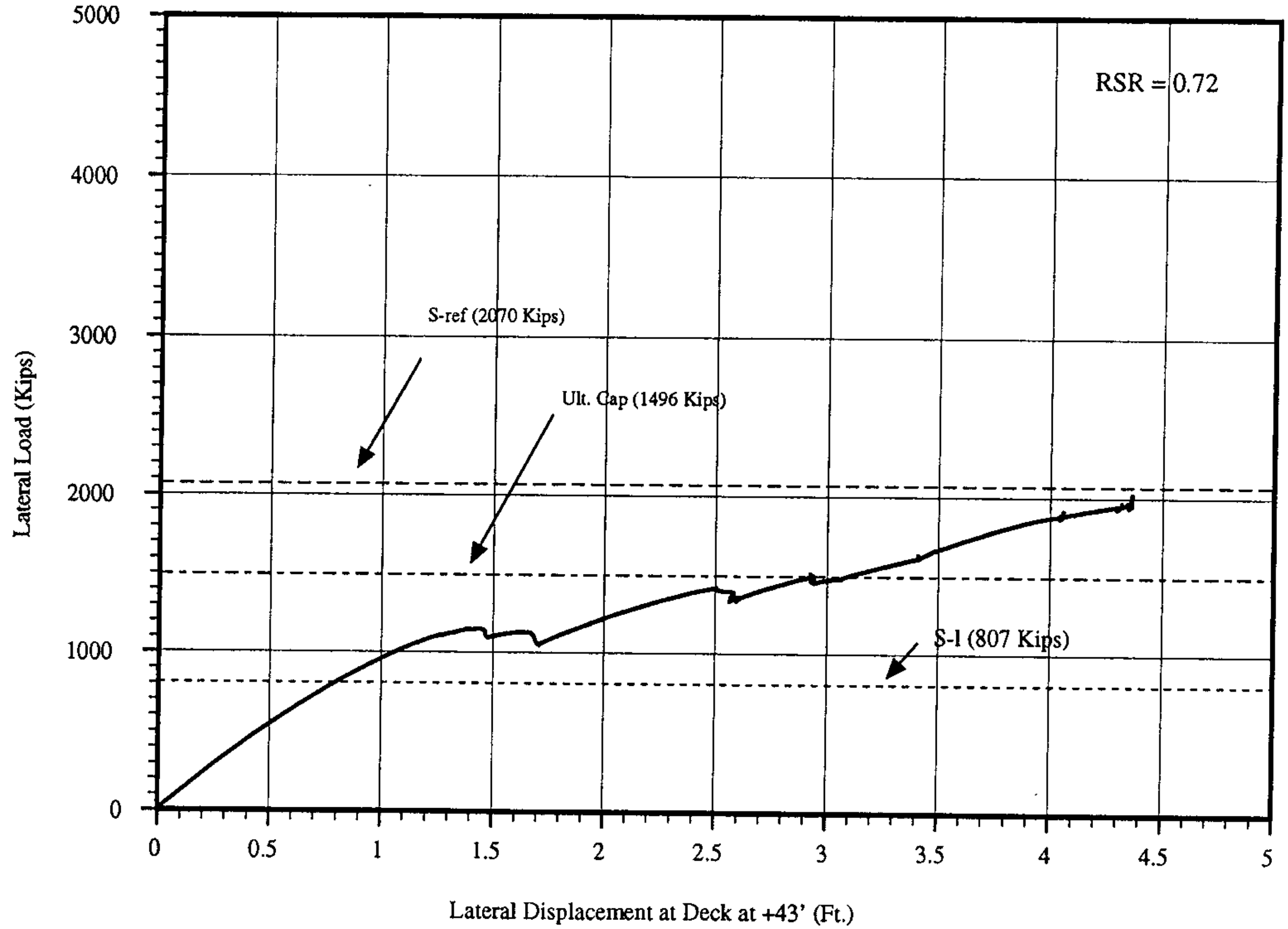


FIGURE A.4.3: Pushover Analysis Results - Diagonal Wave (225° PN)

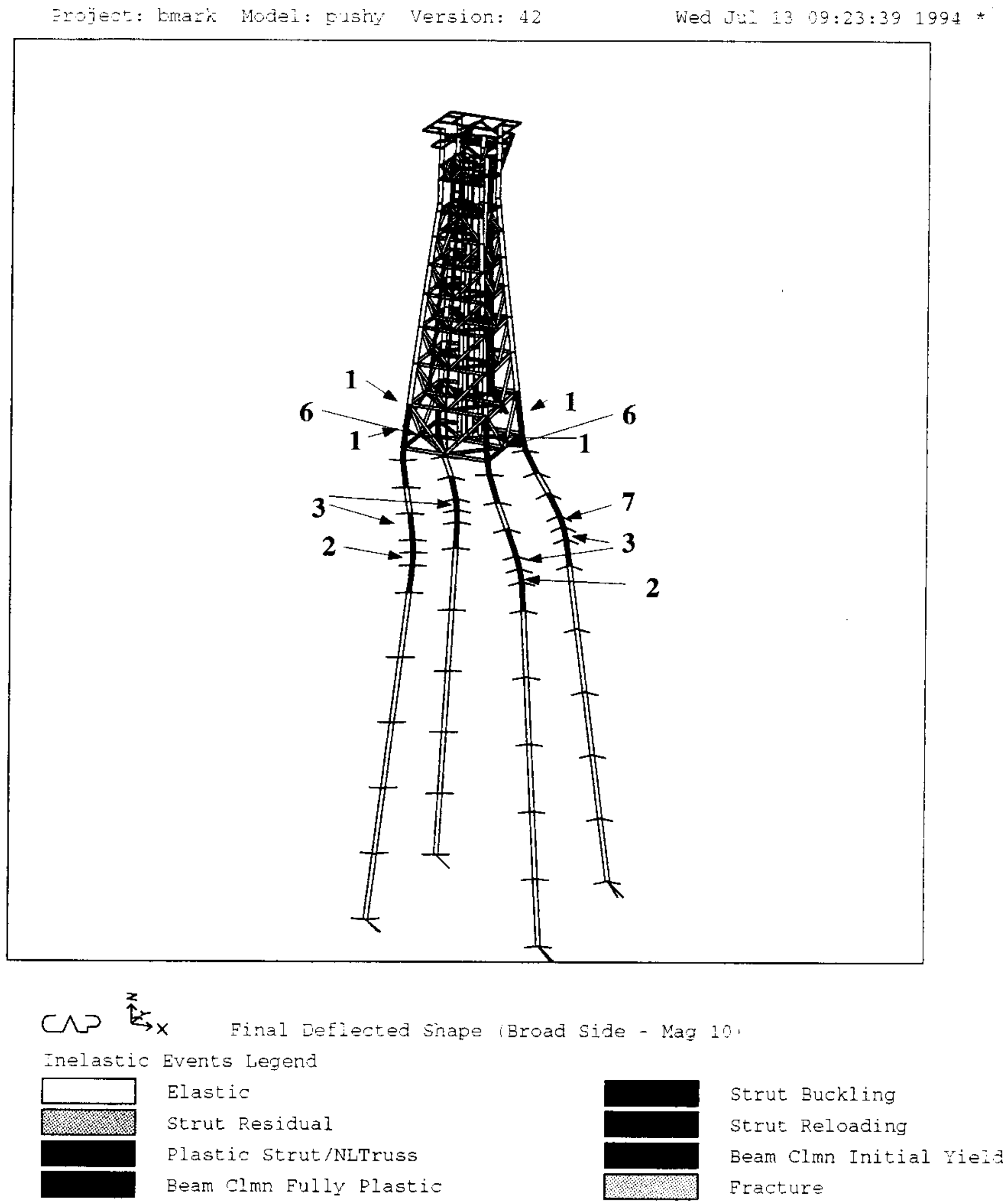


FIGURE A.4.4: Deflected Shape at Analysis Termination and Failure Sequence - Broad-side Wave (270° PN)

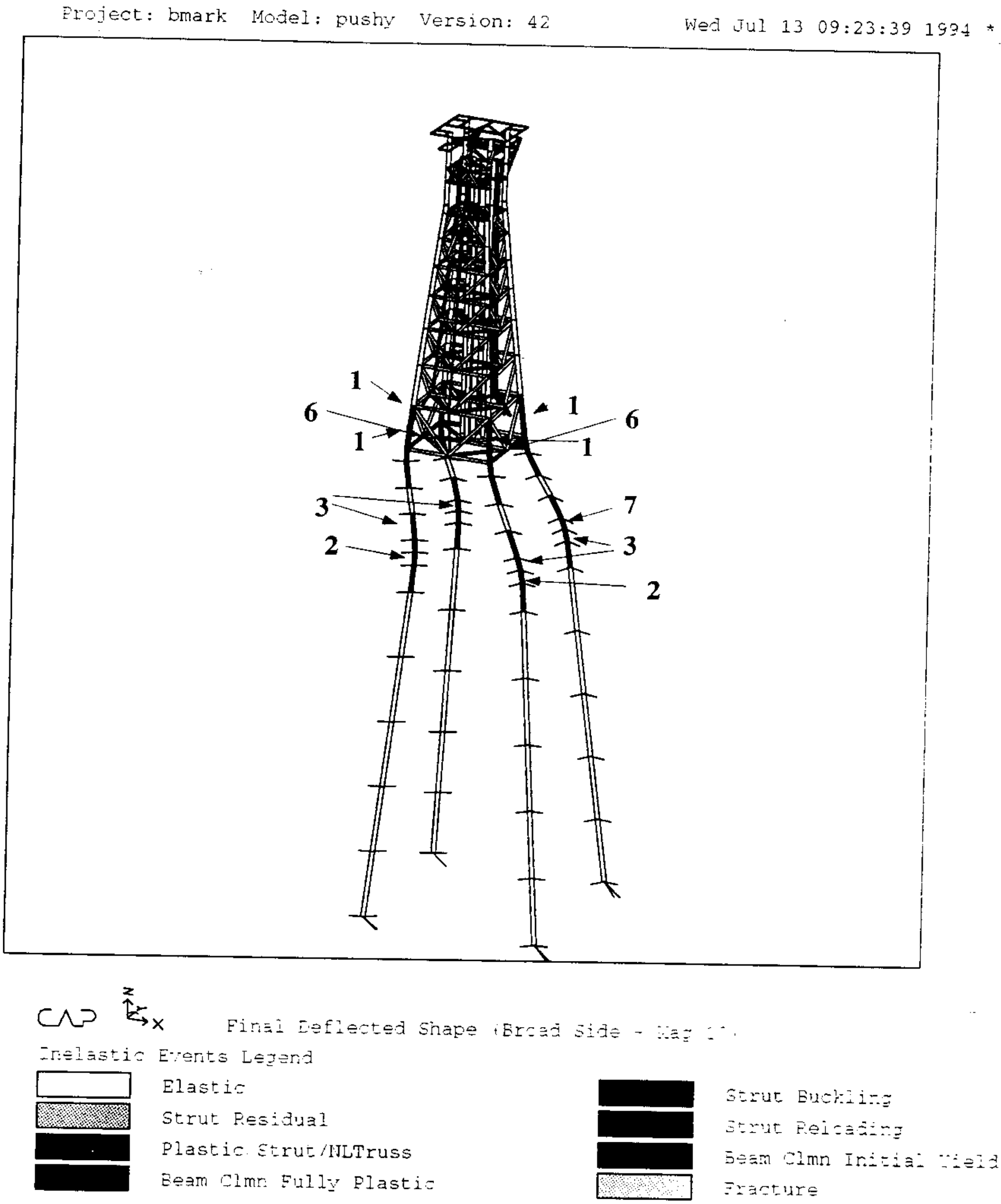


FIGURE A.4.4: Deflected Shape at Analysis Termination and Failure Sequence - Broad-side Wave (270° PN)

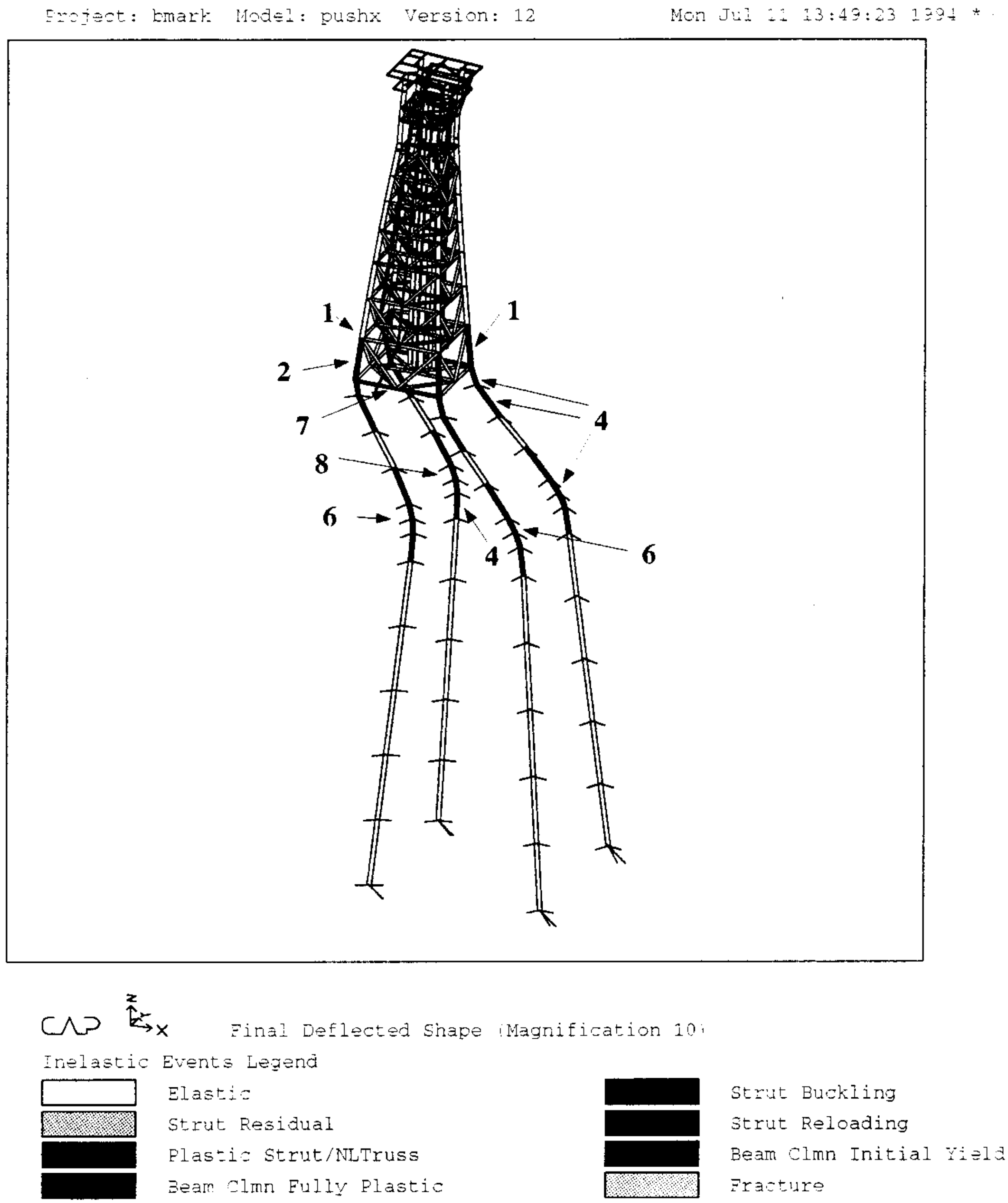


FIGURE A.4.5: Deflected Shape at Analysis Termination and Failure Sequence - End-on Wave (180°)

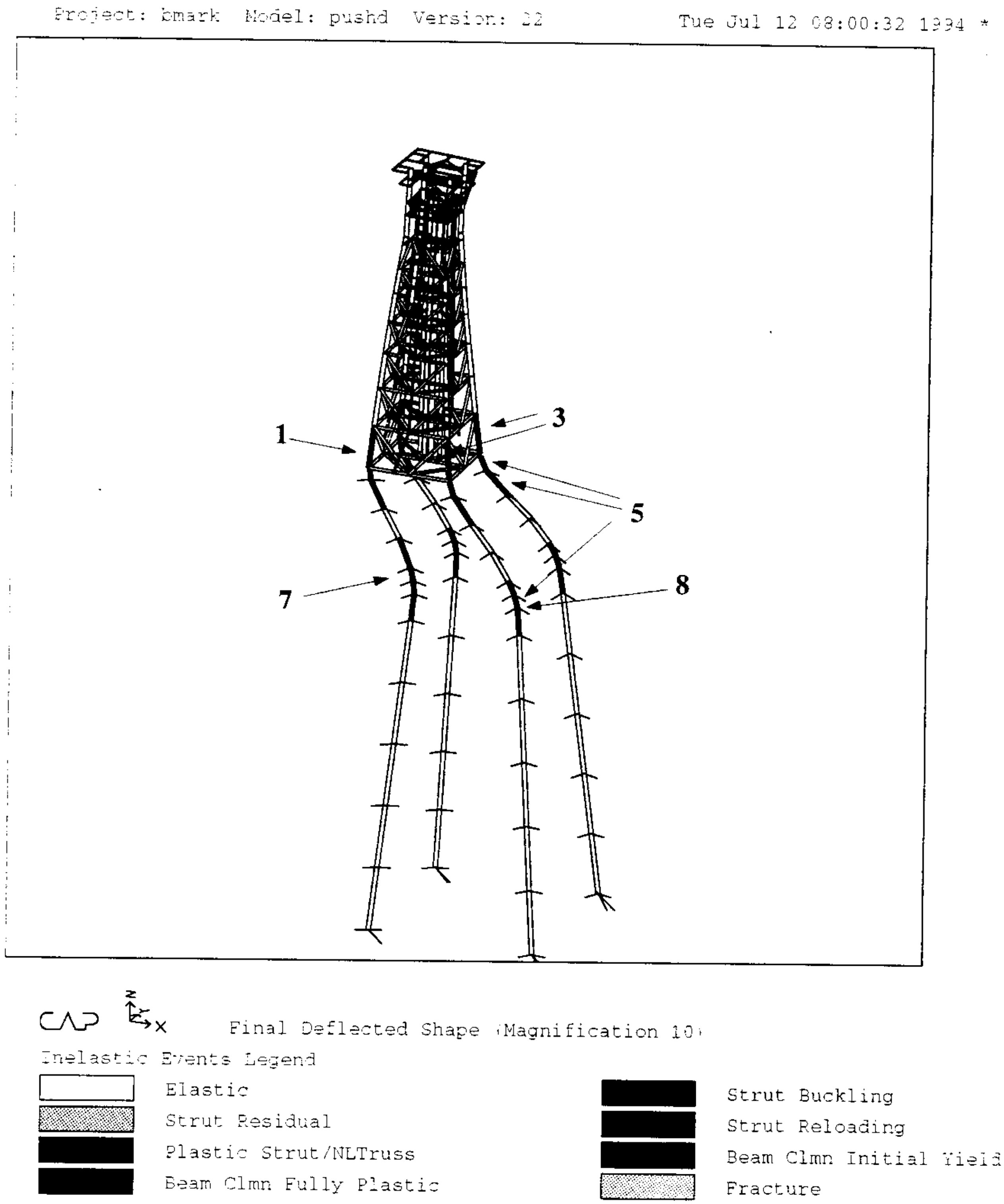


FIGURE A.4.6: Deflected Shape at Analysis Termination and Failure Sequence - Diagonal Wave (225° PN)

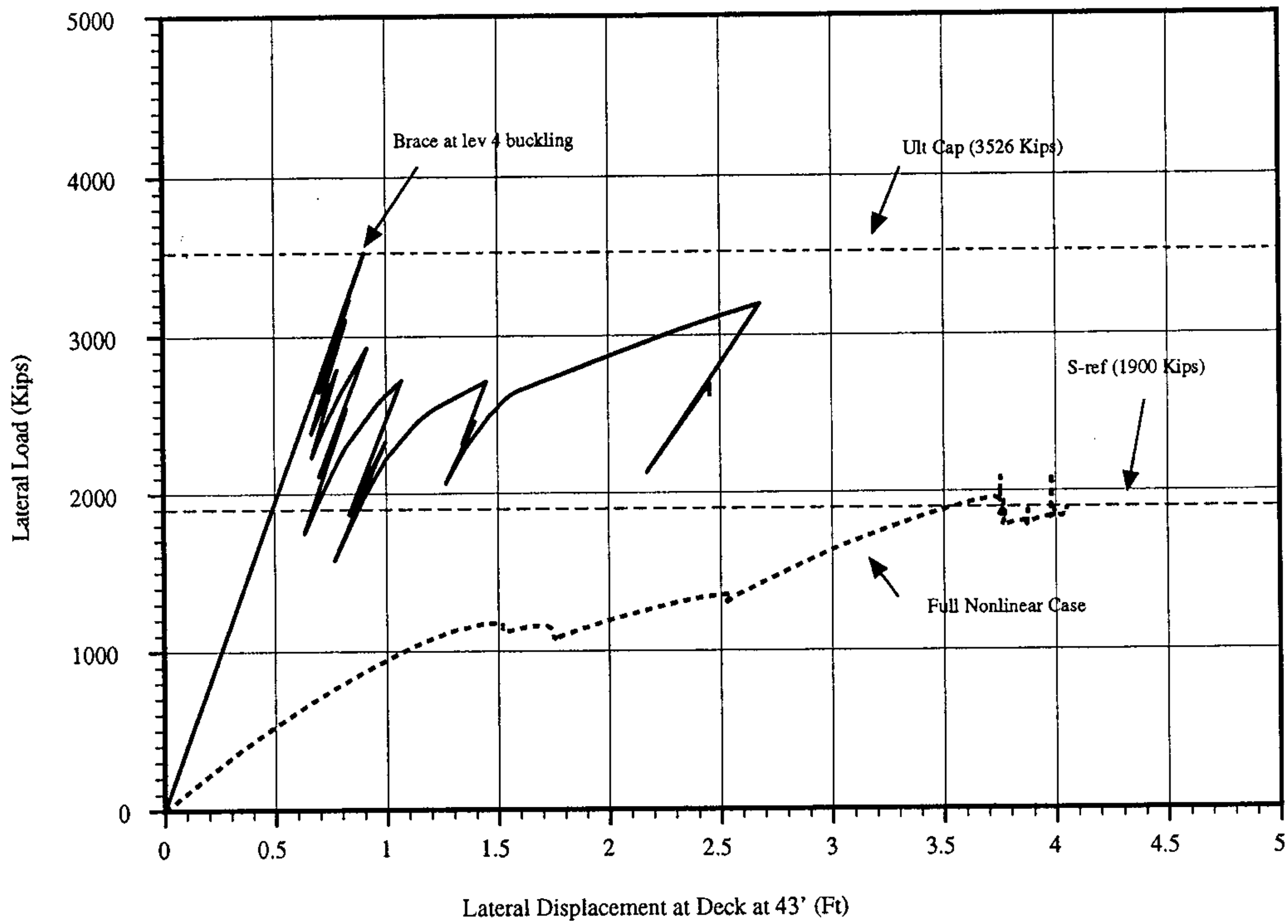


FIGURE A.5.1: Fixed Base Case Results - Broadside Wave (270° PN)

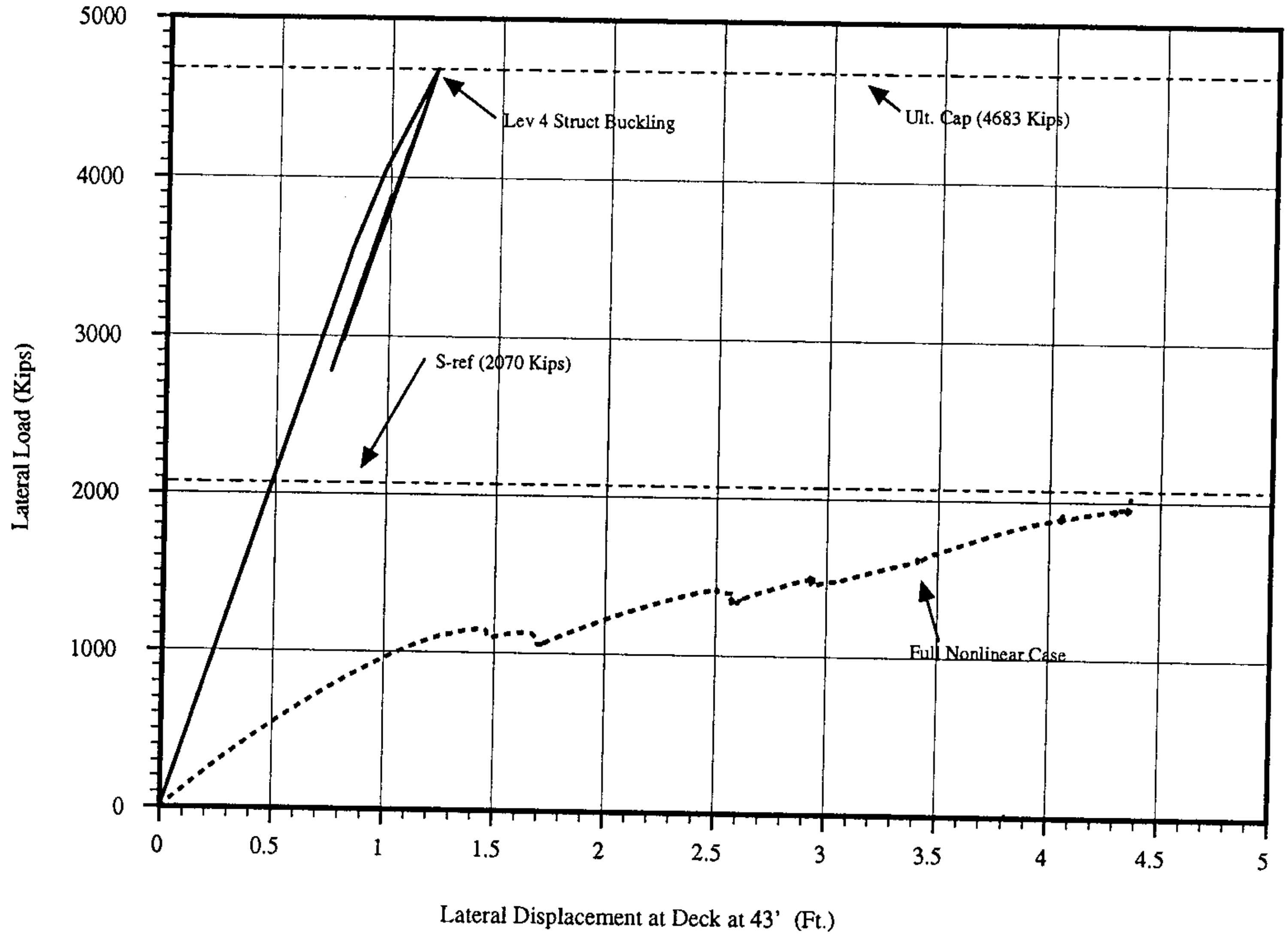


FIGURE A.5.2: Fixed Base Case Results - Diagonal Wave (225° PN)

PARTICIPANT "C"

Benchmark Document Required Information

A: Data for Environmental Loads: Ultimate Capacity Environmental Loads

Number of approach directions	1 of 3	
Orientation w.r.t. Platform North	180 deg	North-South wave
Wave height	61.2 ft	
Wave period	13.5 sec	13.91 sec (apparent)
Current profile	3.89 ft/s	Uniform
Storm surge	3 ft	
Wind speed @ 10m above msl	143.65 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	100 kip	
Wind load	55 kip	
Wave and Current Load	1840 kip	
Total load	1995 kip	

Number of approach directions	2 of 3	
Orientation w.r.t. Platform North	225 deg	Northeast-Southwest wave
Wave height	68 ft	
Wave period	13.5 sec	14.31 sec (apparent)
Current profile	3.89 ft/s	Uniform
Storm surge	3 ft	
Wind speed @ 10m above msl	143.65 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	355 kip	
Wind load	80 kip	
Wave and Current Load	2500 kip	
Total load	2935 kip	

Number of approach directions	3 of 3	
Orientation w.r.t. Platform North	270 deg	East-West wave
Wave height	64.6 ft	
Wave period	13.5 sec	14.04 sec (apparent)
Current profile	3.89 ft/s	Uniform
Storm surge	3 ft	
Wind speed @ 10m above msl	143.65 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	145 kip	
Wind load	60 kip	
Wave and Current Load	2090 kip	
Total load	2295 kip	

Benchmark Document Required Information

A: Data for Environmental Loads: 100-year Environmental Loads

Number of approach directions	1 of 3	
Orientation w.r.t. Platform North	180 deg	North-South wave
Wave height	56.7 ft	
Wave period	13 sec	13.52 sec (apparent)
Current profile	3.55 ft/s	Uniform
Storm surge	3.5 ft	
Wind speed @ 10m above msl	135 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	90 kip	
Wind load	50 kip	
Wave and Current Load	1520 kip	
Total load	1660 kip	

Number of approach directions	2 of 3	
Orientation w.r.t. Platform North	225 deg	Northeast-Southwest wave
Wave height	63 ft	
Wave period	13 sec	13.91 sec (apparent)
Current profile	3.55 ft/s	Uniform
Storm surge	3.5 ft	
Wind speed @ 10m above msl	135 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	165 kip	
Wind load	75 kip	
Wave and Current Load	1990 kip	
Total load	2230 kip	

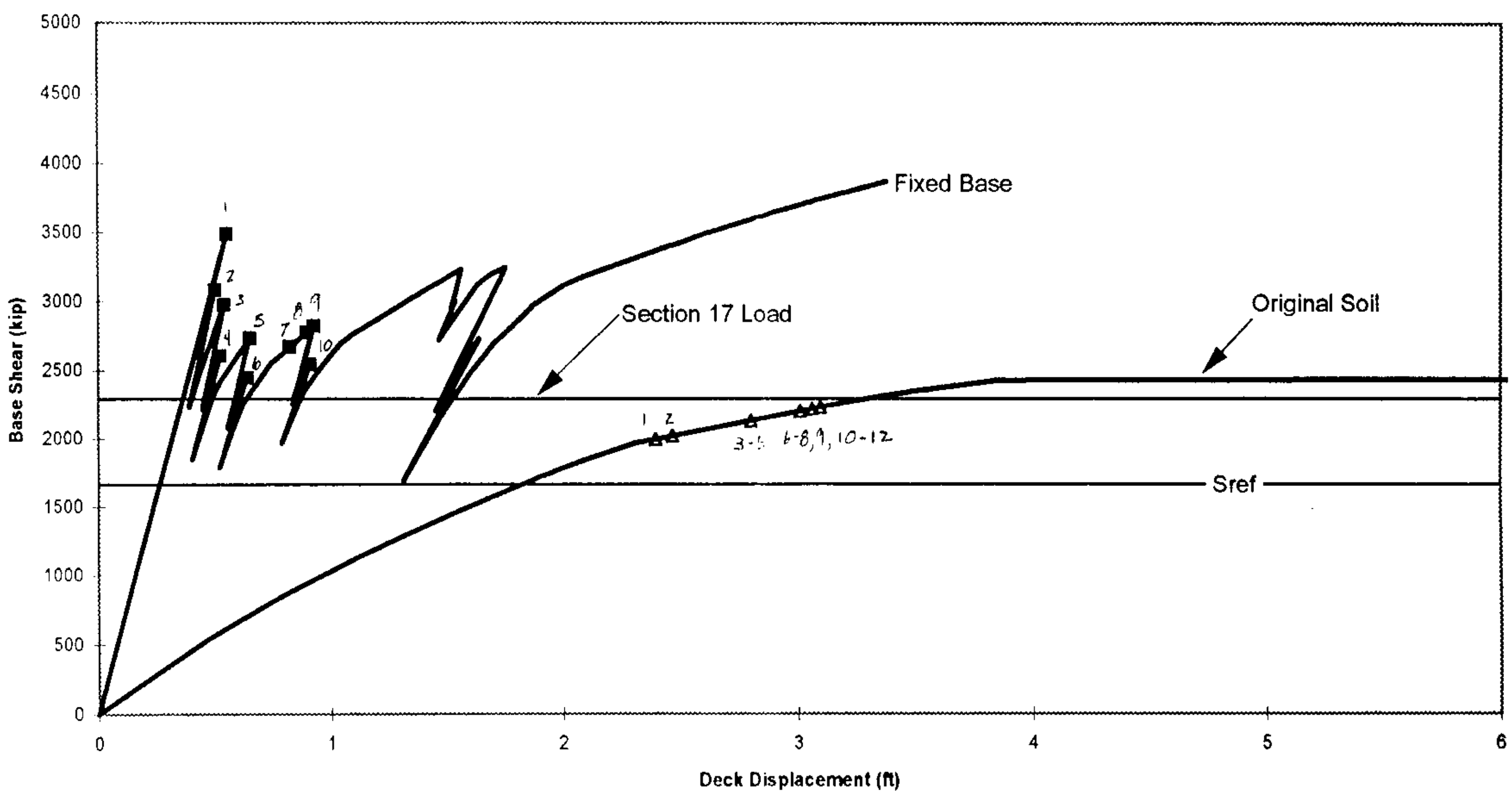
Number of approach directions	3 of 3	
Orientation w.r.t. Platform North	270 deg	East-West wave
Wave height	59.9 ft	
Wave period	13 sec	13.65 sec (apparent)
Current profile	3.55 ft/s	Uniform
Storm surge	3.5 ft	
Wind speed @ 10m above msl	135 ft/s	
Approach used to determine wave/current deck forces	Section 17	C17.6.2
Wave load in deck	120 kip	
Wind load	55 kip	
Wave and Current Load	1685 kip	
Total load	1860 kip	

Benchmark Document Required Information

B. Data for Member Capacity Estimation

Material yield strength (Mean)	42 ksi
Member capacity estimate:	
Braces	(see attached sheets)
Legs/Piles	6075 kip-ft
Piles	5706 kip-ft
Joint capacity estimate	(see attached sheets)
Soil spring (p-y, t-z, q-z) generation	CAP - Automatic

Broadside (N-S) Pushover



	With Foundation Case	Fixed Base Case
Reference Level Load (Sref):	1660 kip	1660 kip
Ultimate Capacity (Ru):	2350 kip	3870 kip
Reserve Strength Ratio (RSR):	1.42	2.33
Numebr of Elements Failed up to Collapse:	21	65
Platform Failure Mode:	Foundation (SOIL)	Leg Bending
Section 17 Load:	2000 kip	2000 kip

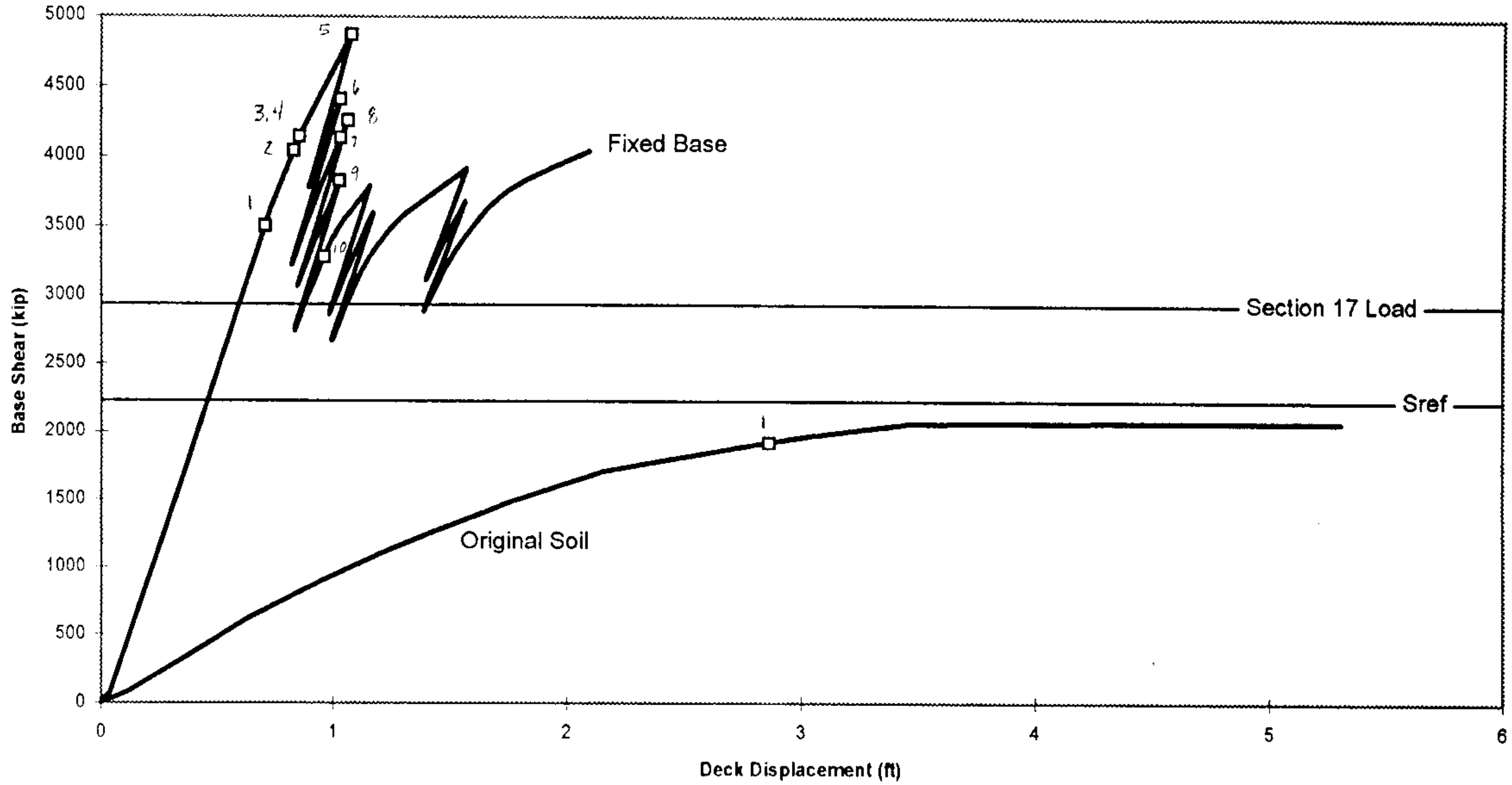
Broadside (N-S) Pushover

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	PileA2-611	26	2.39	1990	Beam Column	Beam Clmn Initial Yield
2	PileA2-610	27	2.46	2015	Beam Column	Beam Clmn Initial Yield
3	LegB1-1	29	2.80	2130	Beam Column	Beam Clmn Initial Yield
4	PileB2-810	29	2.80	2130	Beam Column	Beam Clmn Initial Yield
5	PileB2-811	29	2.80	2130	Beam Column	Beam Clmn Initial Yield
6	LegA2-11	30	3.01	2200	Beam Column	Beam Clmn Initial Yield
7	PileA2-609	30	3.01	2200	Beam Column	Beam Clmn Initial Yield
8	PileA2-622	30	3.01	2200	Beam Column	Beam Clmn Initial Yield
9	LegB2-12	31	3.06	2220	Beam Column	Beam Clmn Initial Yield
10	LegA1-11	33	3.09	2230	Beam Column	Beam Clmn Initial Yield
11	PileB1-1210	33	3.09	2230	Beam Column	Beam Clmn Initial Yield
12	PileB1-1211	33	3.09	2230	Beam Column	Beam Clmn Initial Yield

Broadside (N-S) Pushover, Fixed Base

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	Diag06-2	9	0.56	3490	Strut	Strut Buckling
2	Diag06-6	14	0.50	3085	Strut	Strut Buckling
3	Diag05-6	30	0.54	2975	Strut	Strut Buckling
4	Diag05-1	43	0.53	2605	Strut	Strut Buckling
5	Diag04-6	68	0.65	2735	Strut	Strut Buckling
6	Diag04-1	82	0.64	2445	Strut	Strut Buckling
7	LegA2-9	100	0.82	2670	Beam Column	Beam Clmn Initial Yield
8	LegB1-9	101	0.90	2775	Beam Column	Beam Clmn Initial Yield
9	Diag03-6	105	0.93	2820	Strut	Strut Buckling
10	Diag03-1	118	0.91	2540	Strut	Strut Buckling

Diagonal (NE-SW) Pushover



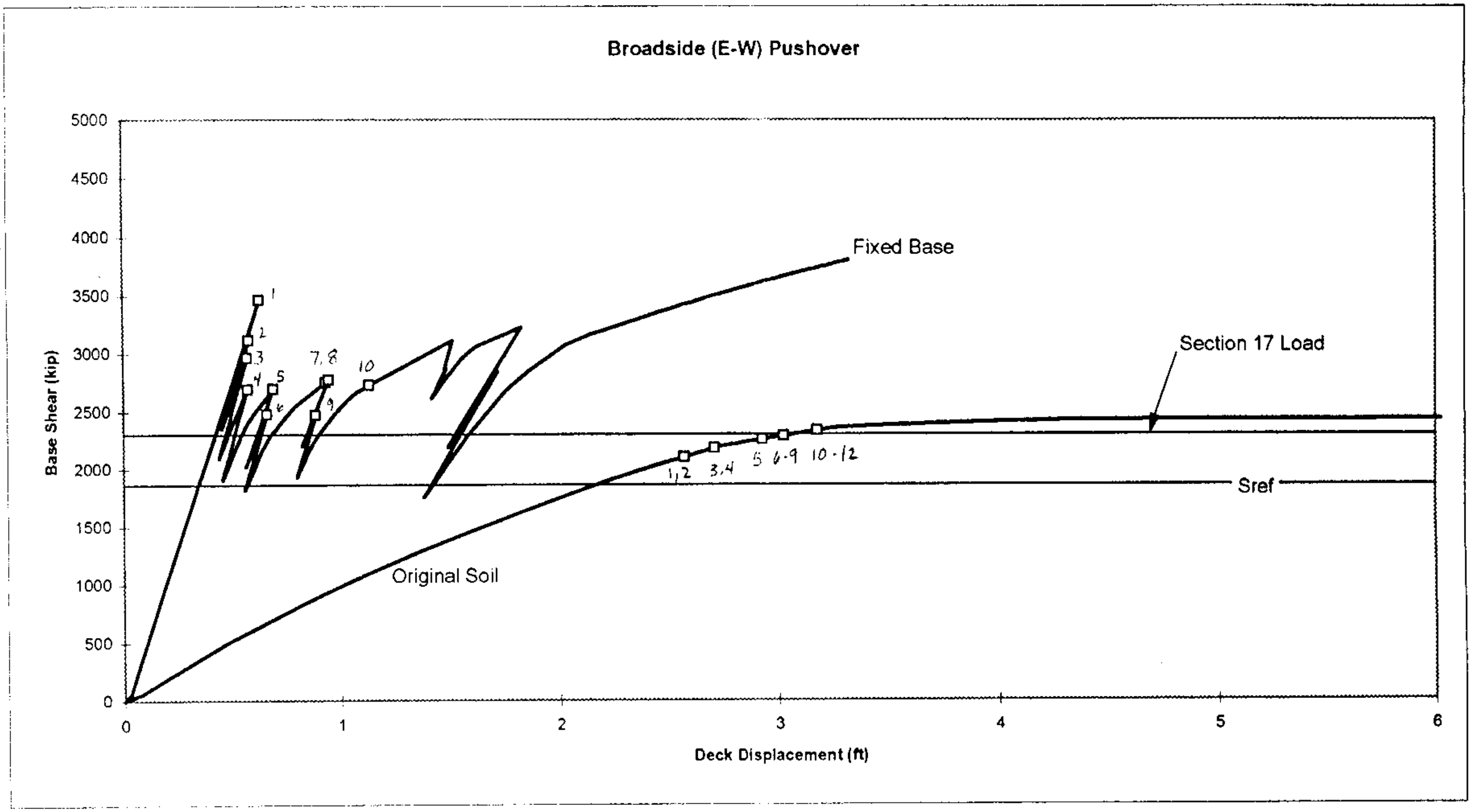
	With Foundation Case	Fixed Base Case
Reference Level Load (Sref):	2230 kip	2230 kip
Ultimate Capacity (Ru):	2070 kip	4870 kip
Reserve Strength Ratio (RSR):	0.93	2.18
Numebr of Elements Failed up to Collapse:	1	65
Platform Failure Mode:	Foundation (Soil)	Leg Bending
Section 17 Load:	2935 kip	2935 kip

Diagonal (NE-SW) Pushover

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	LegB1-1	27	2.86	1920	Beam Column	Beam Clmn Initial Yield

Diagonal (NE-SW) Pushover, Fixed Base

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	LegA2-9	4	0.71	3505	Beam Column	Beam Clmn Initial Yield
2	LegB1-9	37	0.83	4045	Beam Column	Beam Clmn Initial Yield
3	LegA2-11	44	0.86	4145	Beam Column	Beam Clmn Initial Yield
4	LegB1-11	44	0.86	4145	Beam Column	Beam Clmn Initial Yield
5	Diag05-7	108	1.08	4870	Strut	Strut Buckling
6	Diag06-7	121	1.03	4410	Strut	Strut Buckling
7	Lev06-8	130	1.03	4125	Beam Column	Beam Clmn Initial Yield
8	Diag06-2	134	1.06	4255	Strut	Strut Buckling
9	Diag05-1	148	1.03	3830	Strut	Strut Buckling
10	Lev06-5	158	0.96	3280	Beam Column	Beam Clmn Fully Plastic



	With Foundation Case	Fixed Base Case
Reference Level Load (Sref):	1860 kip	1860 kip
Ultimate Capacity (Ru):	2430 kip	3800 kip
Reserve Strength Ratio (RSR):	1.31	2.04
Numebr of Elements Failed up to Collapse:	18	58
Platform Failure Mode:	Foundation (SOIL)	Leg Bending
Section 17 Load:	2300 kip	2300 kip

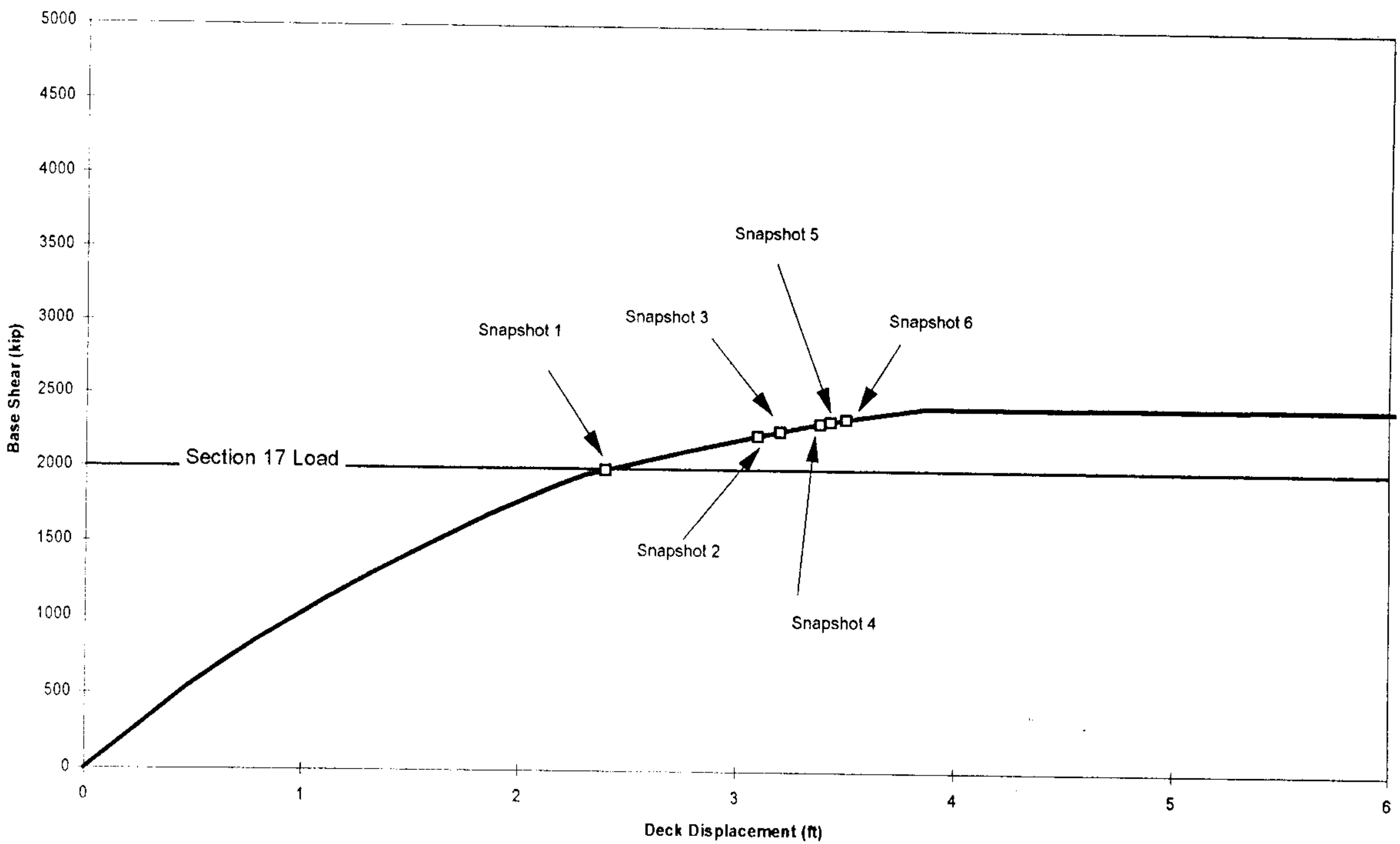
Broadside (E-W) Pushover

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	PileA1-1010	12	2.57	2100	Beam Column	Beam Clmn Initial Yield
2	PileA1-1011	12	2.57	2100	Beam Column	Beam Clmn Initial Yield
3	PileA2-610	14	2.70	2180	Beam Column	Beam Clmn Initial Yield
4	PileA2-611	14	2.70	2180	Beam Column	Beam Clmn Initial Yield
5	LegA1-11	19	2.92	2255	Beam Column	Beam Clmn Initial Yield
6	PileA1-1009	21	3.02	2285	Beam Column	Beam Clmn Initial Yield
7	PileA1-1022	21	3.02	2285	Beam Column	Beam Clmn Initial Yield
8	PileB1-1210	21	3.02	2285	Beam Column	Beam Clmn Initial Yield
9	PileB1-1211	21	3.02	2285	Beam Column	Beam Clmn Initial Yield
10	LegA2-11	23	3.17	2330	Beam Column	Beam Clmn Initial Yield
11	PileA2-609	23	3.17	2330	Beam Column	Beam Clmn Initial Yield
12	PileA2-622	23	3.17	2330	Beam Column	Beam Clmn Initial Yield

Broadside (E-W) Pushover, Fixed Base

	Member Name	First Load Step	Deck Displacement (ft)	Base Shear (kip)	Element Type	Worst Event Description
1	Diag06-7	9	0.63	3465	Strut	Strut Buckling
2	Diag05-7	20	0.58	3120	Strut	Strut Buckling
3	Diag06-4	36	0.57	2965	Strut	Strut Buckling
4	Diag05-4	54	0.57	2690	Strut	Strut Buckling
5	Diag04-7	79	0.69	2700	Strut	Strut Buckling
6	Diag04-4	94	0.66	2475	Strut	Strut Buckling
7	LegA2-9	111	0.93	2755	Beam Column	Beam Clmn Initial Yield
8	Diag03-7	114	0.95	2775	Strut	Strut Buckling
9	Diag03-4	126	0.89	2470	Strut	Strut Buckling
10	LegB1-9	144	1.13	2730	Beam Column	Beam Clmn Initial Yield

Broadside (N-S) Pushover
Original Soil



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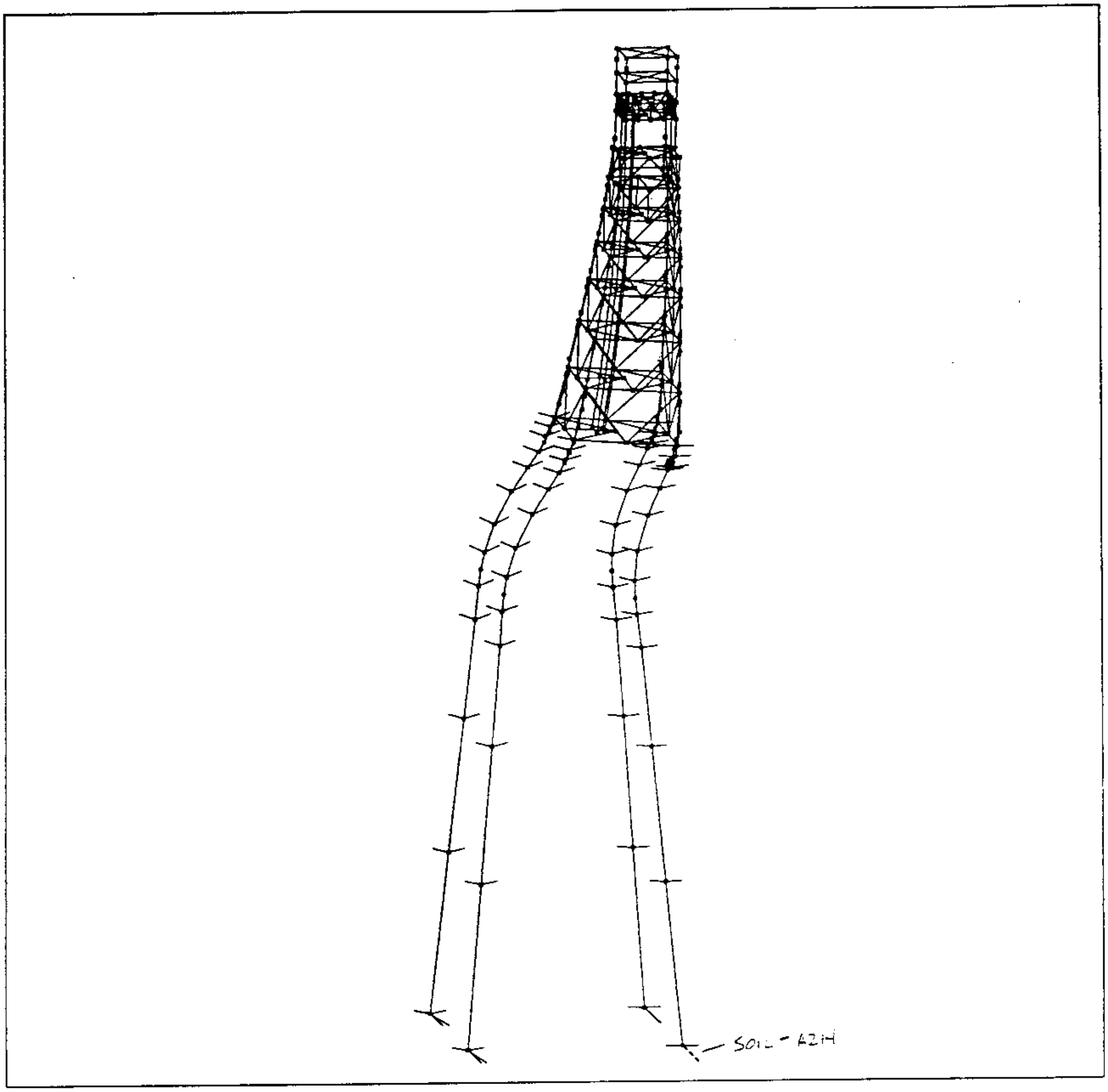

CAP

Broadside (N-S) Push - snapshot 1

Inelastic Events Legend

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

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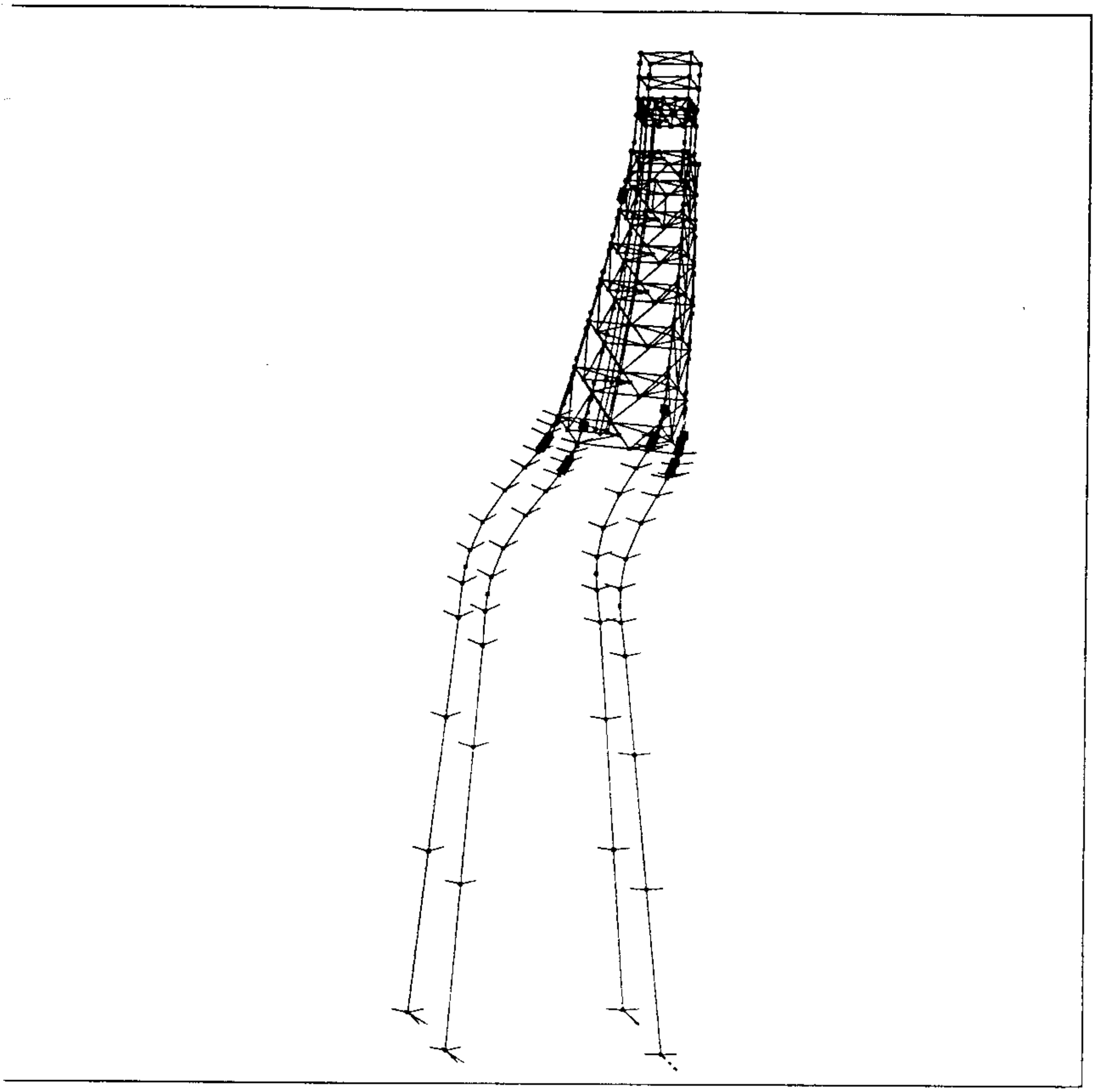


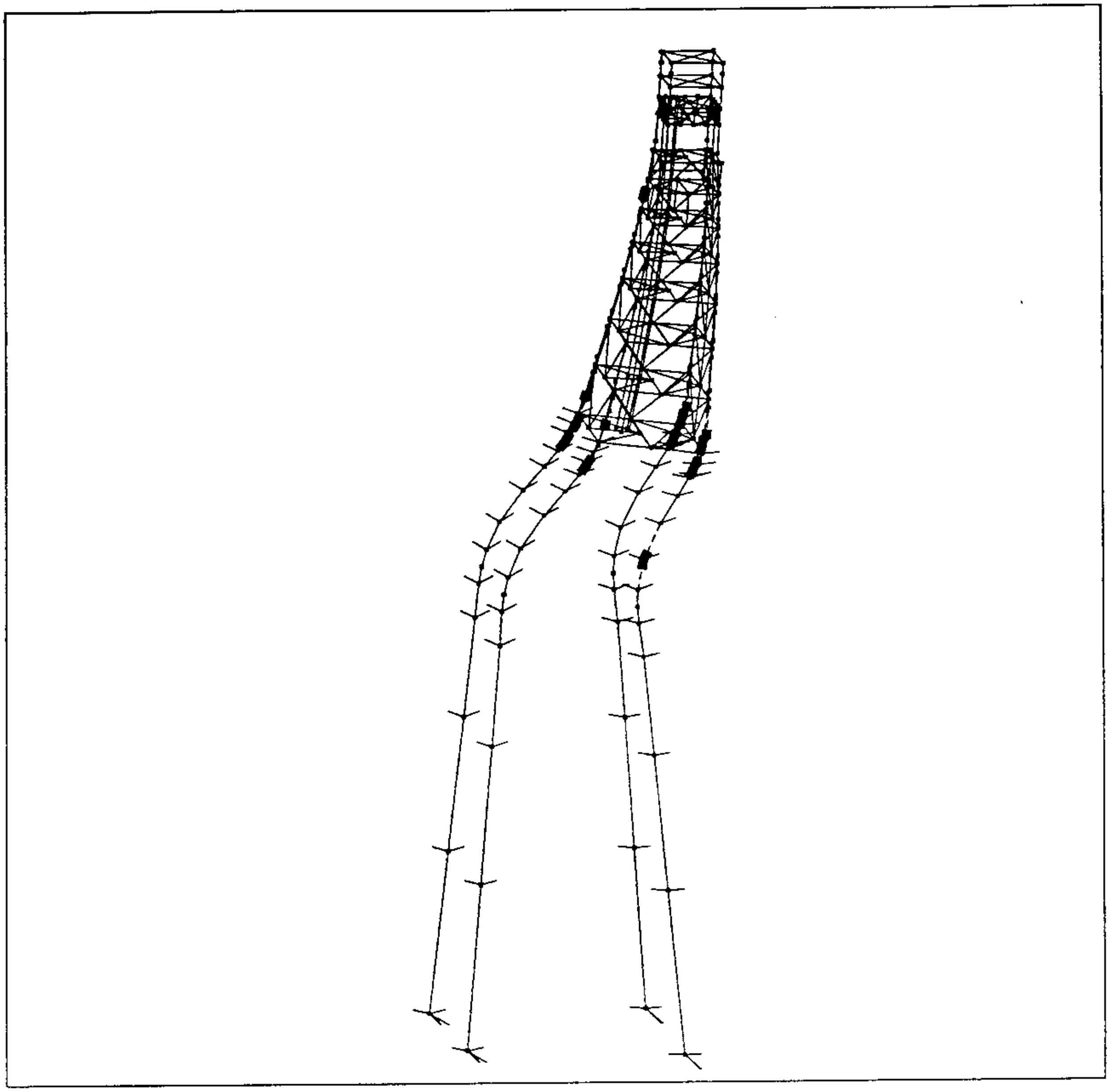
CAP

Broadside (N-S) Push - snapshot 3

Inelastic Events Legend

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

Project: Benchmark Model: 225 Version:Sat Aug 13 15:43:18 1994 *

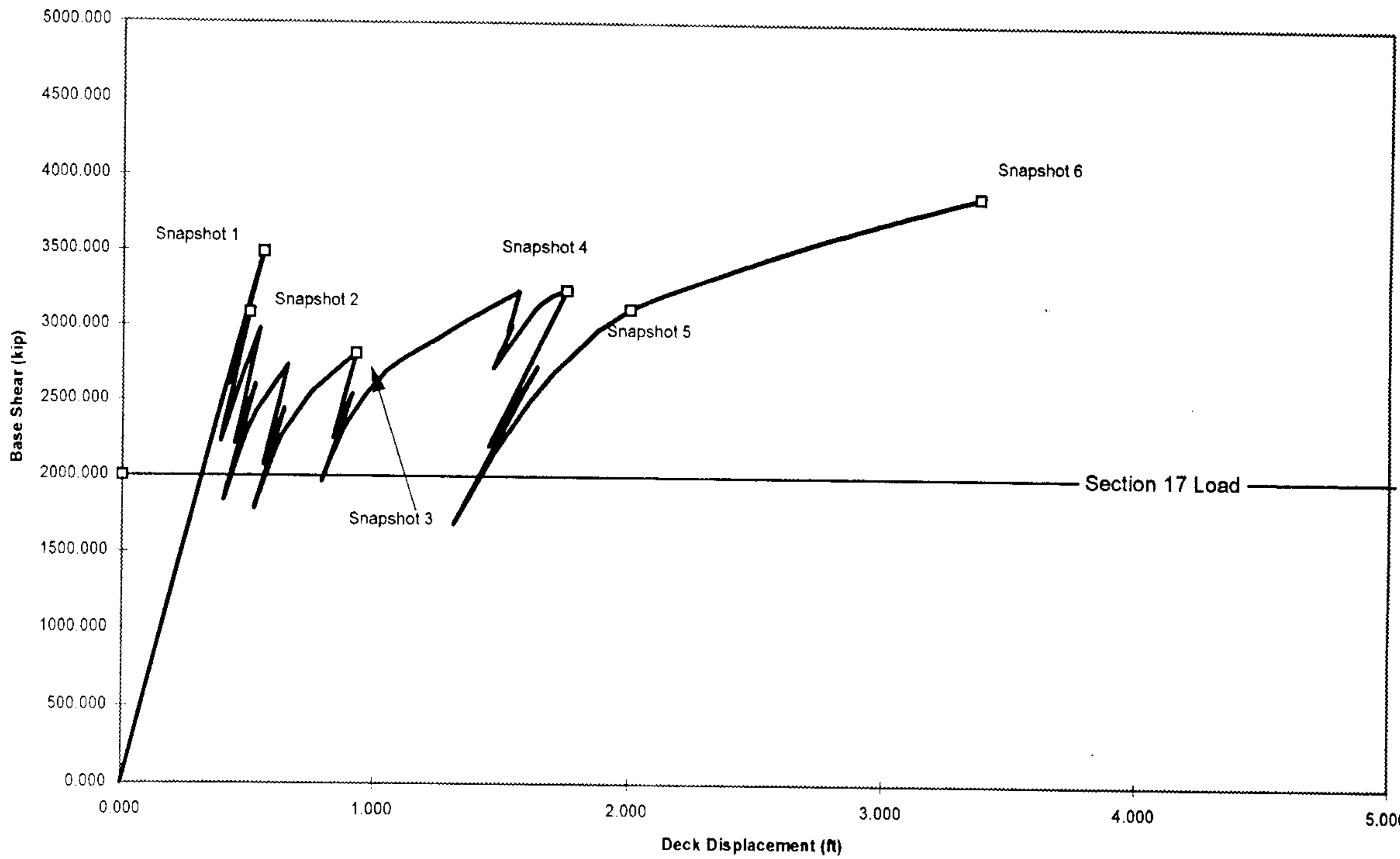


CAP Broadside (N-S) Push - snapshot 6

Inelastic Events Legend

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

Broadside (N-S) Push Fixed Base



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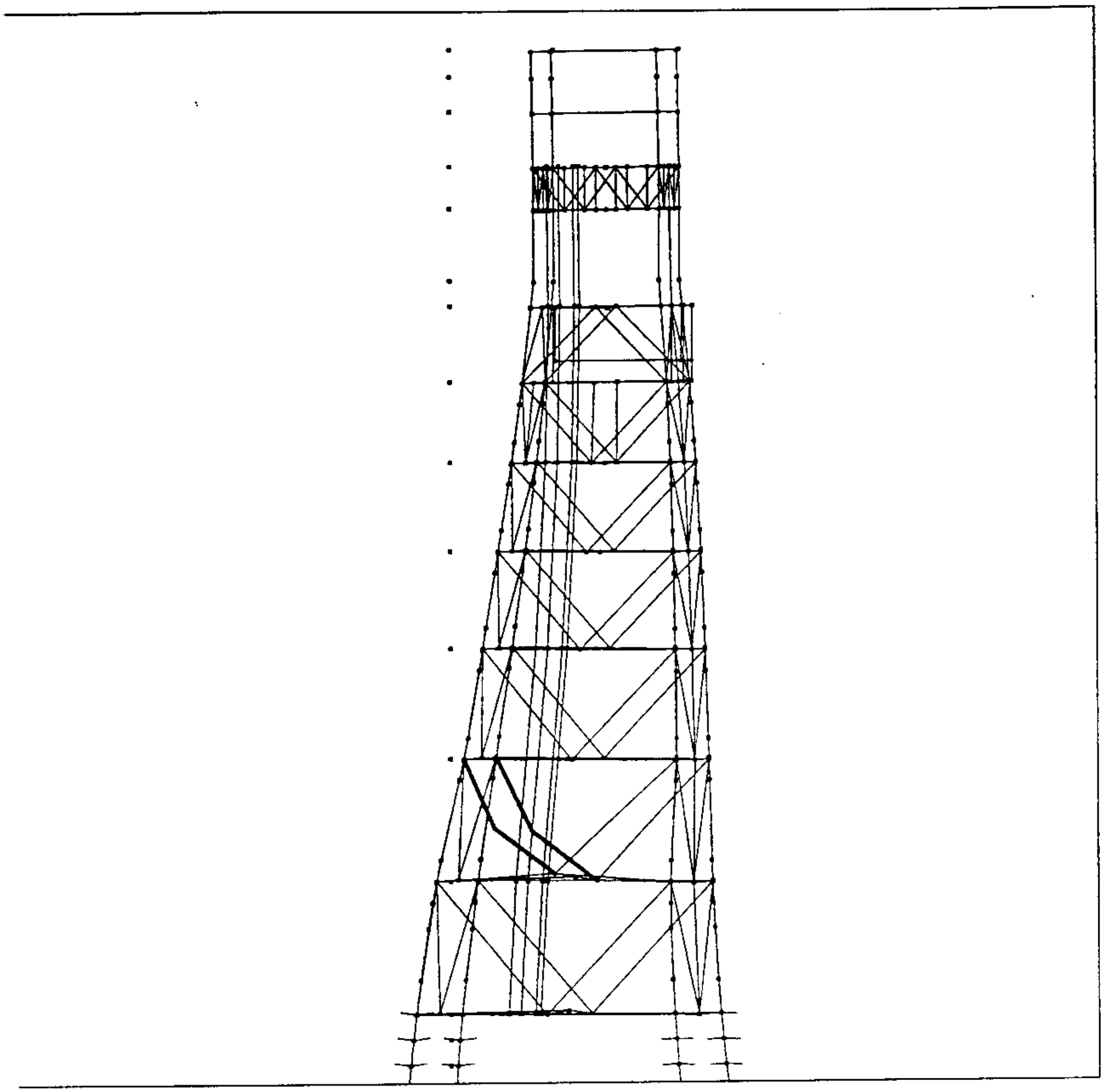
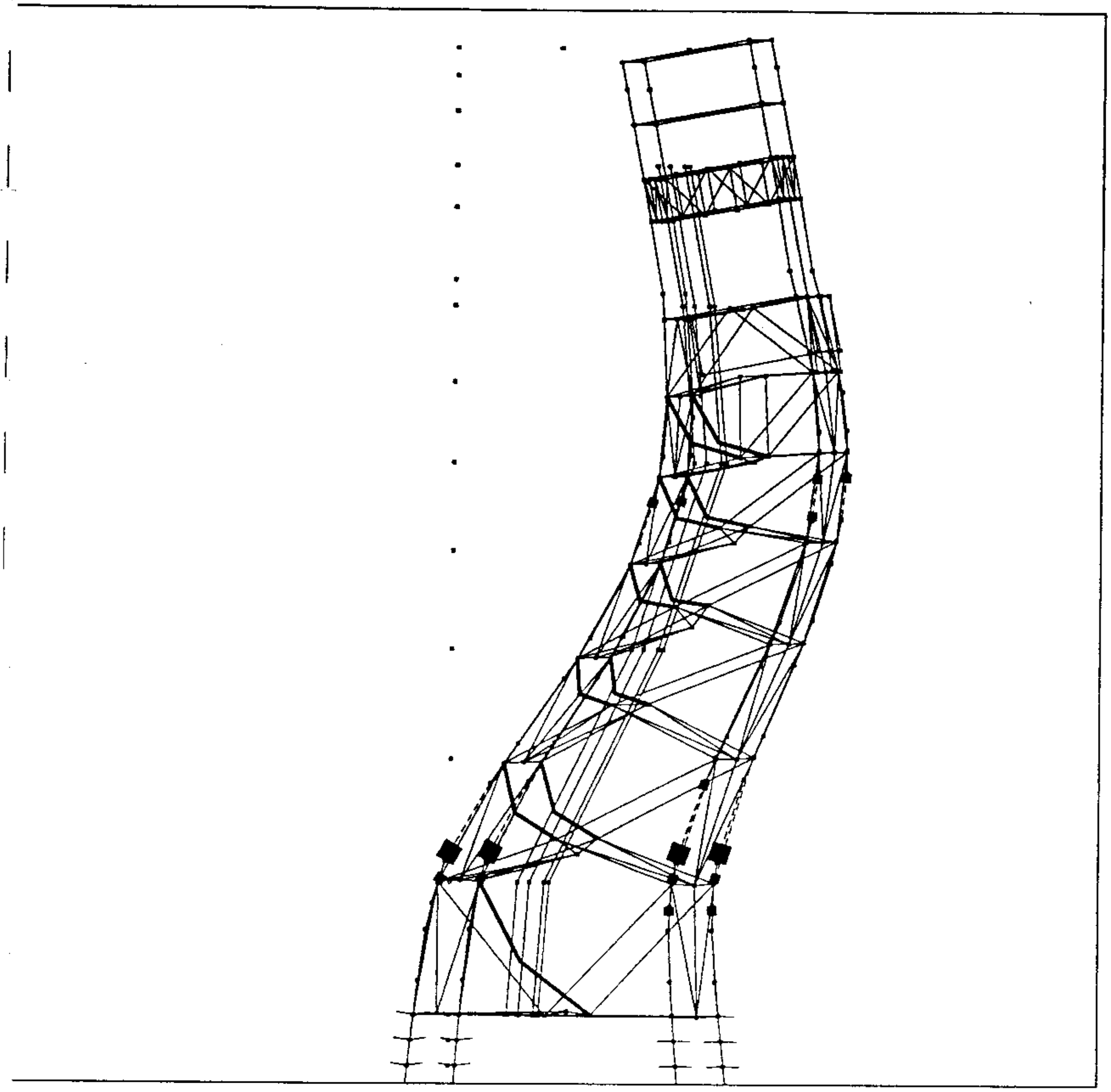


CAP Broadside (N-S) Push, Fixed Base - snapshot 2

Inelastic Events Legend

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

Project: Benchmark Model: 225-C-Fixed Version: 1Sat Aug 13 17:01:24 1994 *



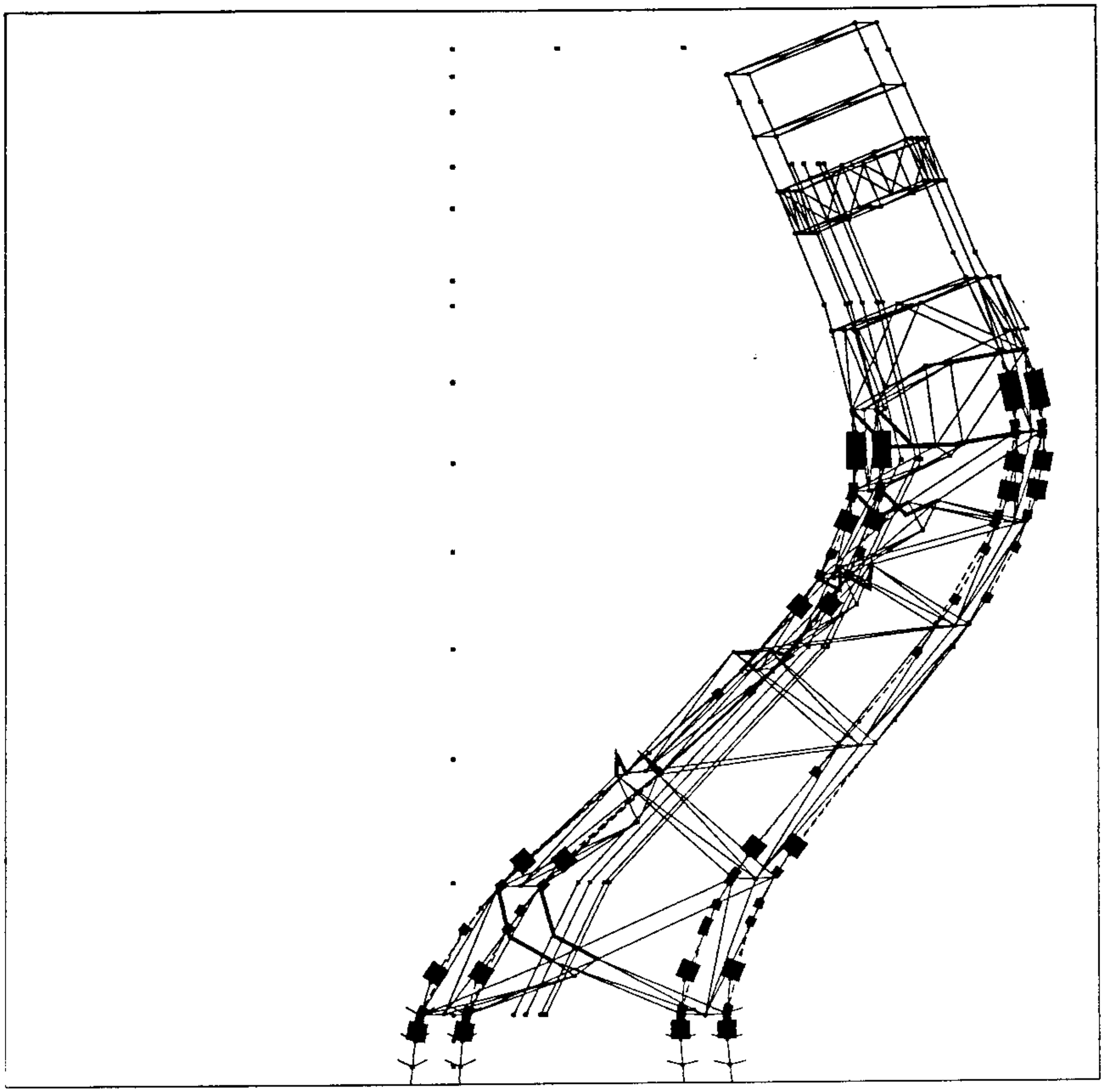
CAP

Broadside (N-S) Push, Fixed Base - snapshot 4

Inelastic Events Legend

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

Project: Benchmark Model: 225-C-Fixed Version: 1Sat Aug 13 17:02:04 1994 *

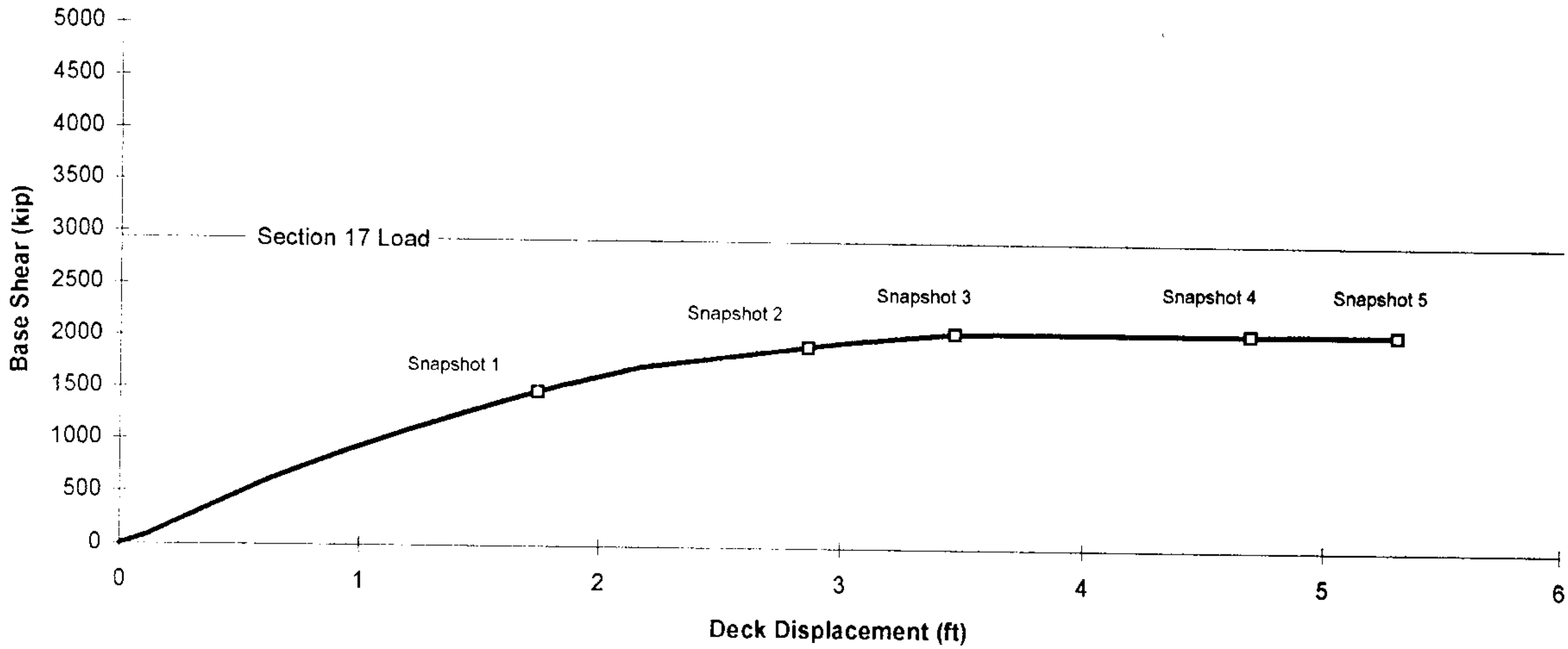


CAP Broadside (N-S) Push, Fixed Base - snapshot 6

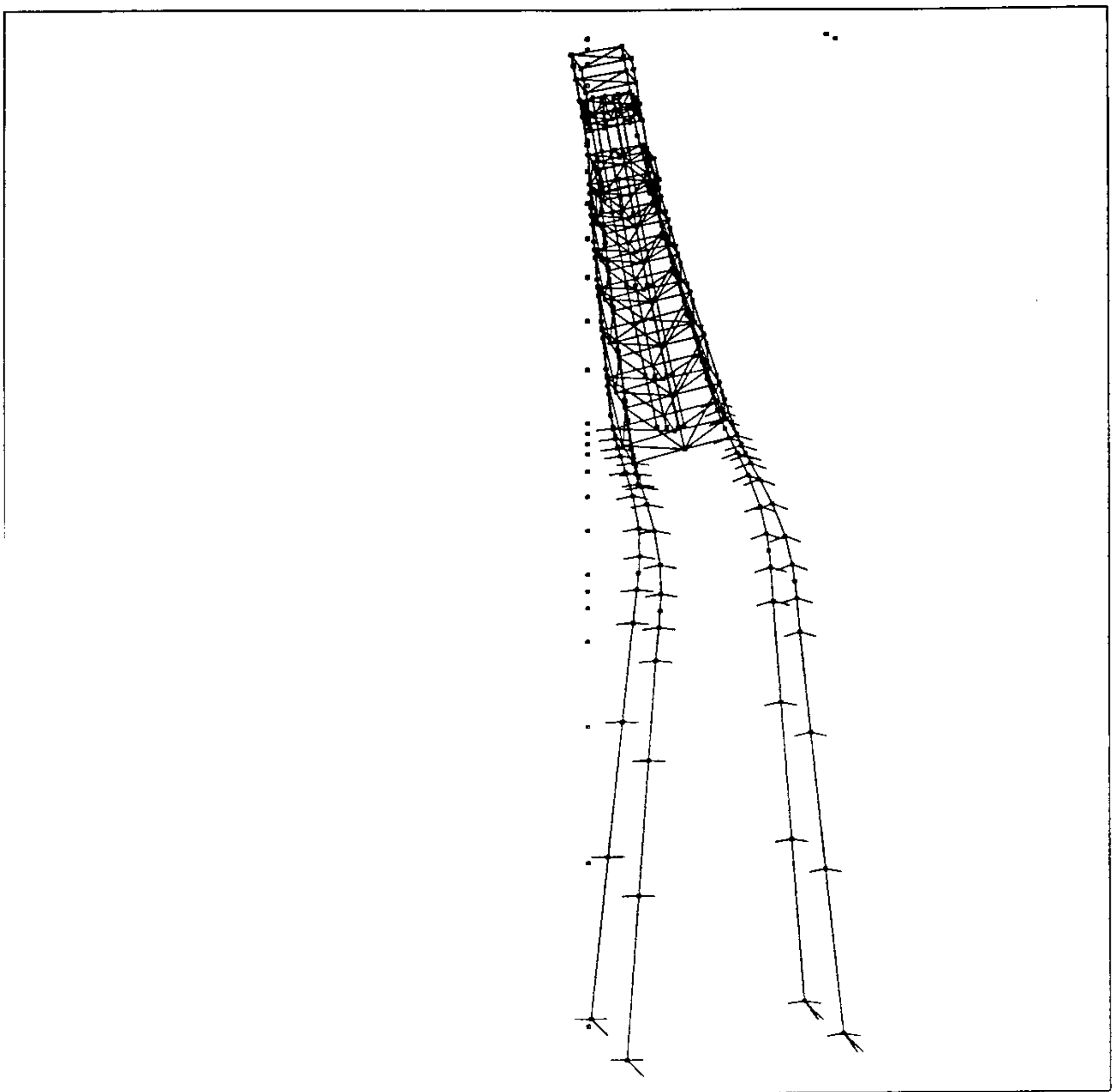
Inelastic Events Legend

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

Diagonal (NE-SW) Direction
Original Soils



Project: Benchmark Model: 270 : Version:Fri Aug 12 17:32:07 1994 *



CAP z x Diagonal Pushover - snapshot 5

Inelastic Events Legend

Elastic

Strut Residual

Plastic Strut/NLTruss

Beam Clmn Fully Plastic

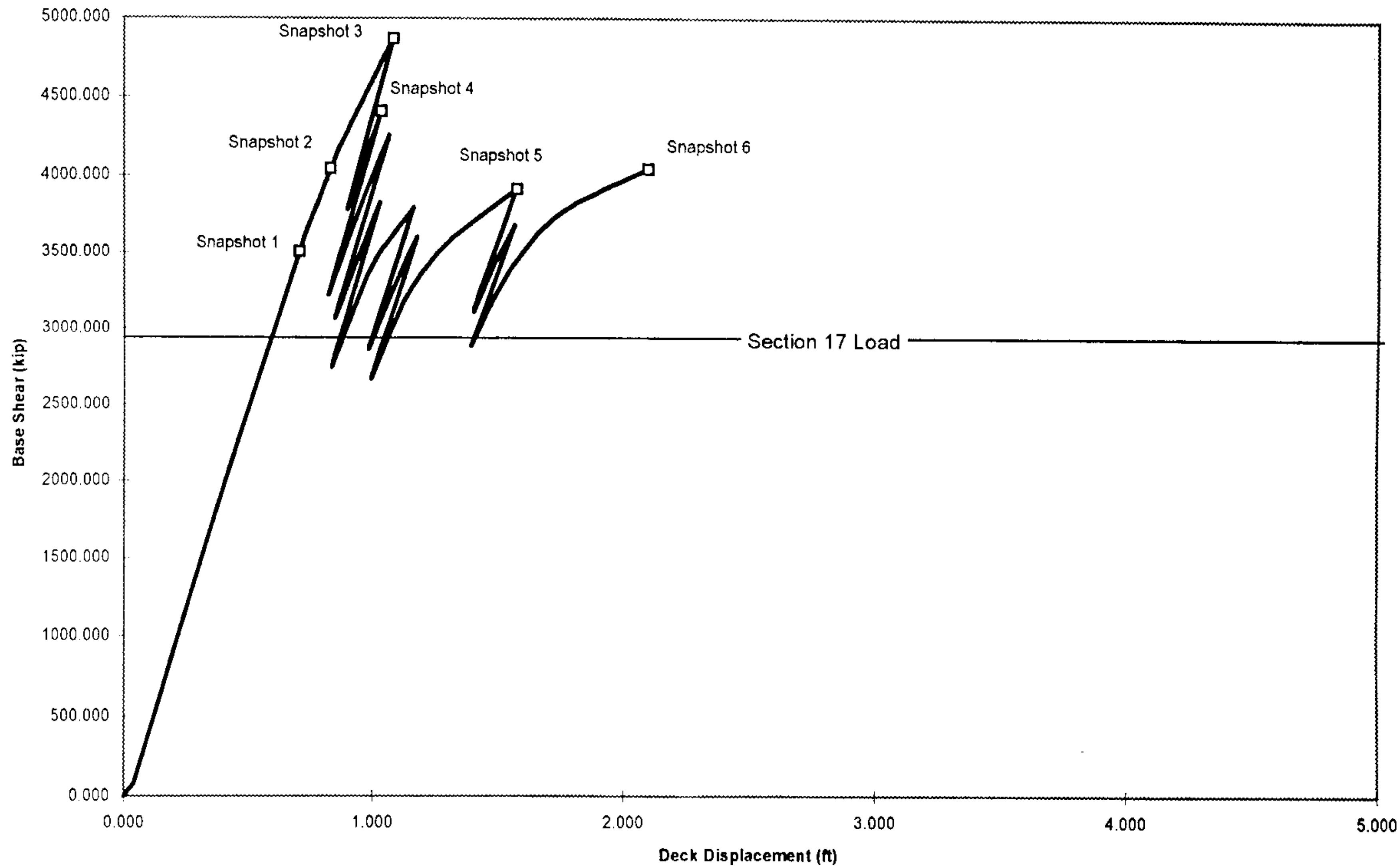
Strut Buckling

Strut Reloading

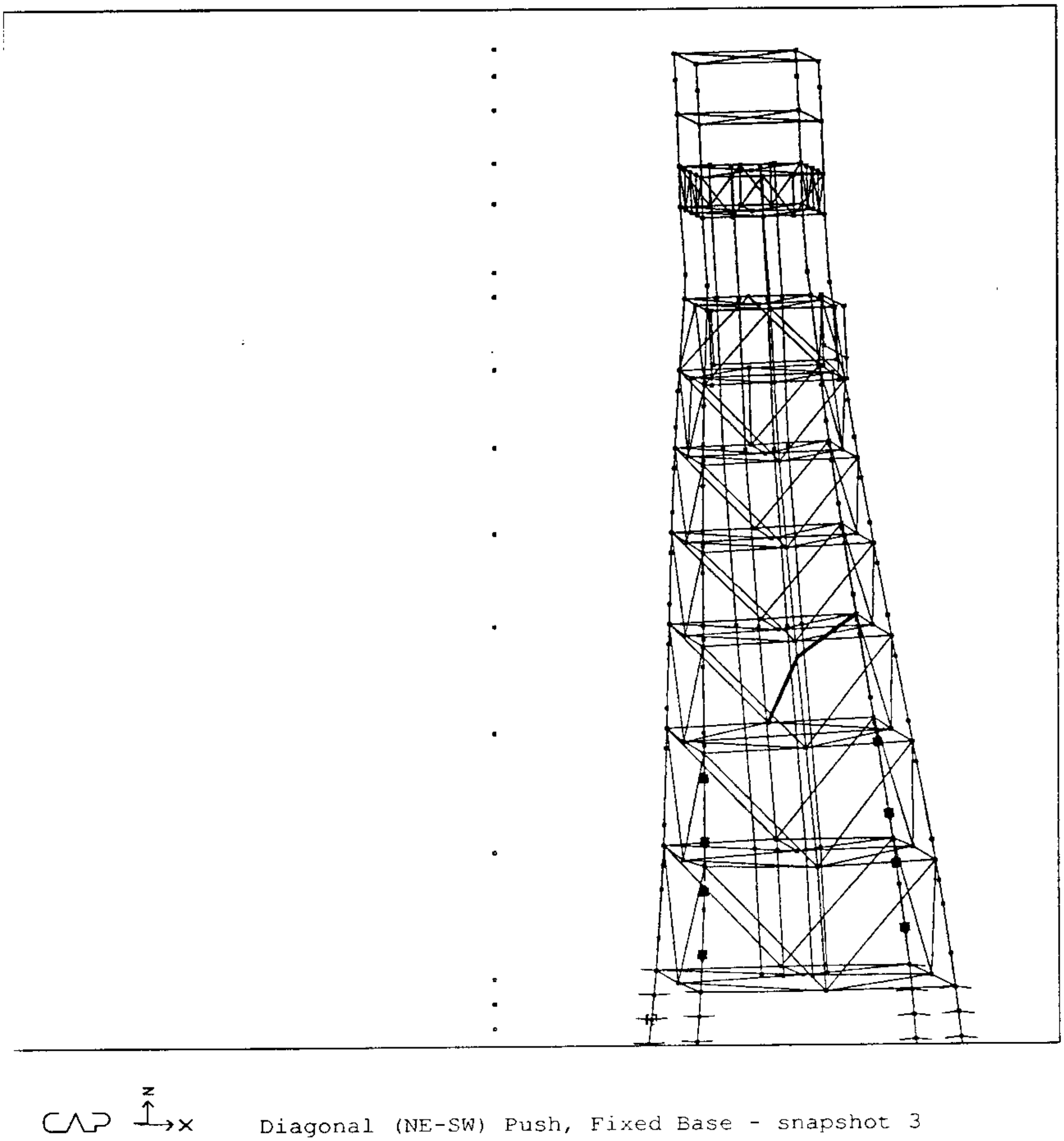
Beam Clmn Initial Yield

Fracture

Diagonal (NE-SW) Pushover Fixed Base



Project: Benchmark Model: 270-C-Fixed Version: 1Sat Aug 13 17:25:02 1994 *

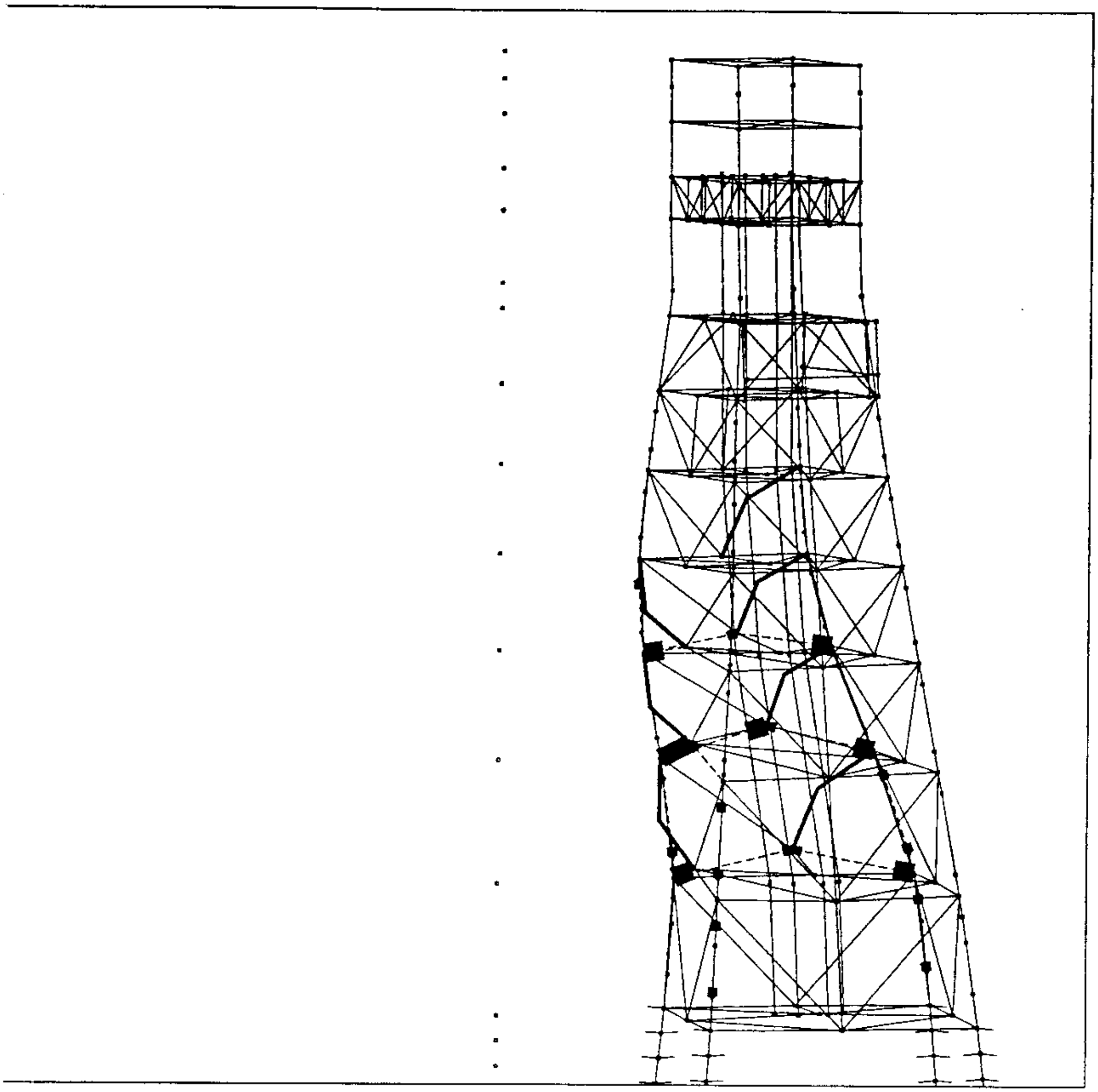


CAP Diagonal (NE-SW) Push, Fixed Base - snapshot 3

Inelastic Events Legend

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

Project: Benchmark Model: 270-C-Fixed Version: 1Sat Aug 13 17:27:14 1994 *

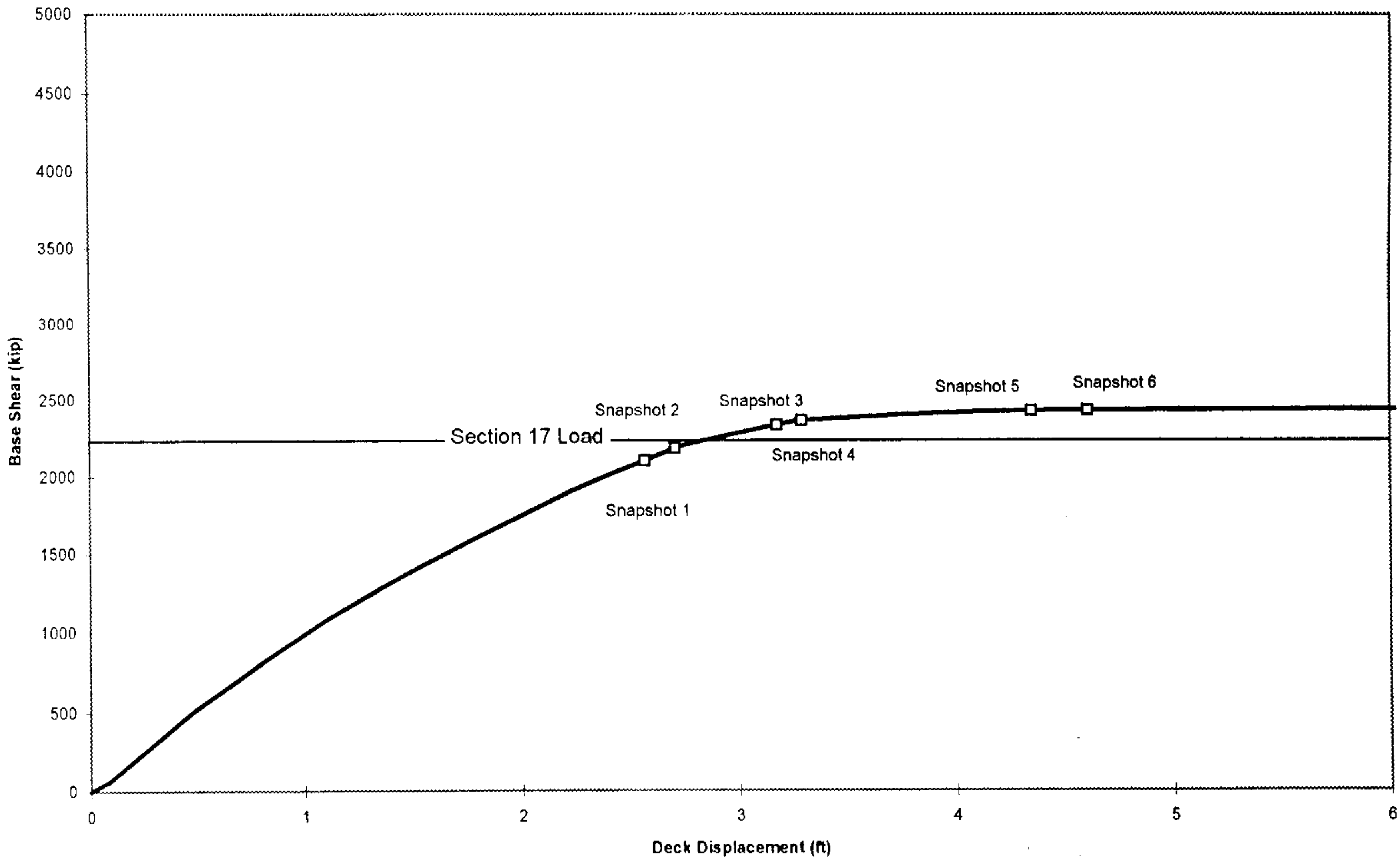


CAP z x Diagonal (NE-SW) Push, Fixed Base - snapshot 5

Inelastic Events Legend

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

Broadside (E-W) Pushover



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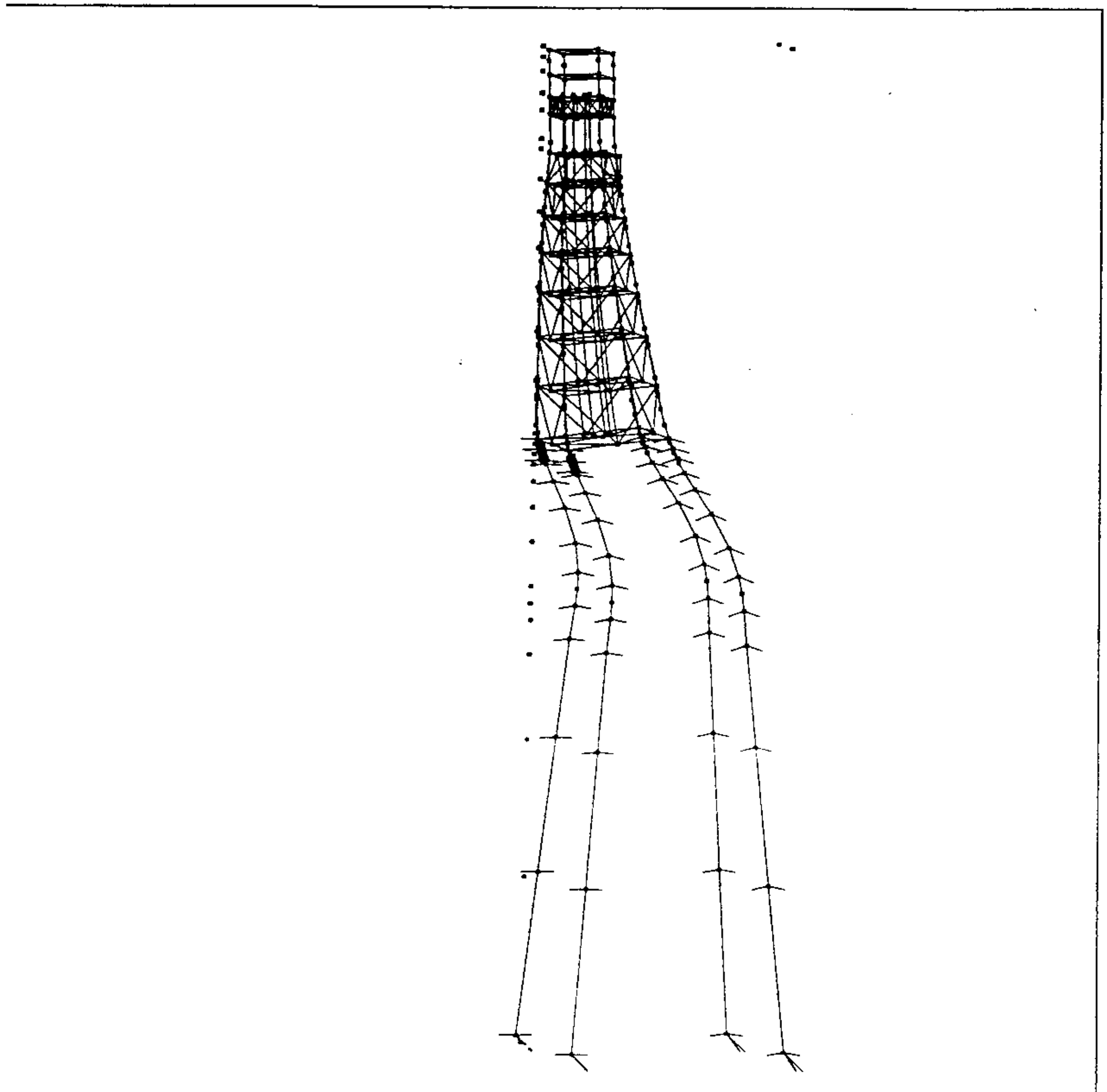
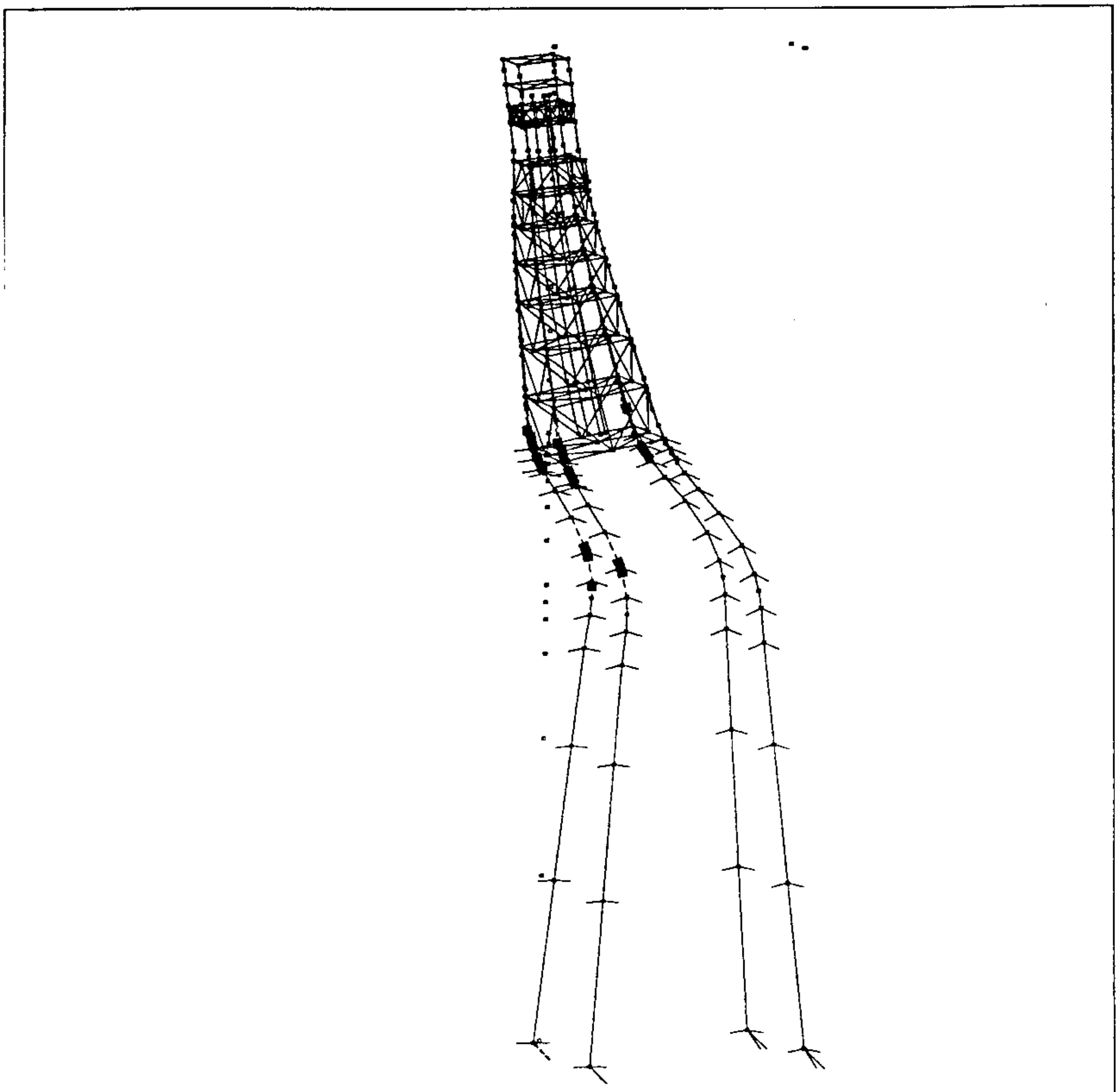


CAP Broadside (E-W) Push - snapshot 2

Inelastic Events Legend

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

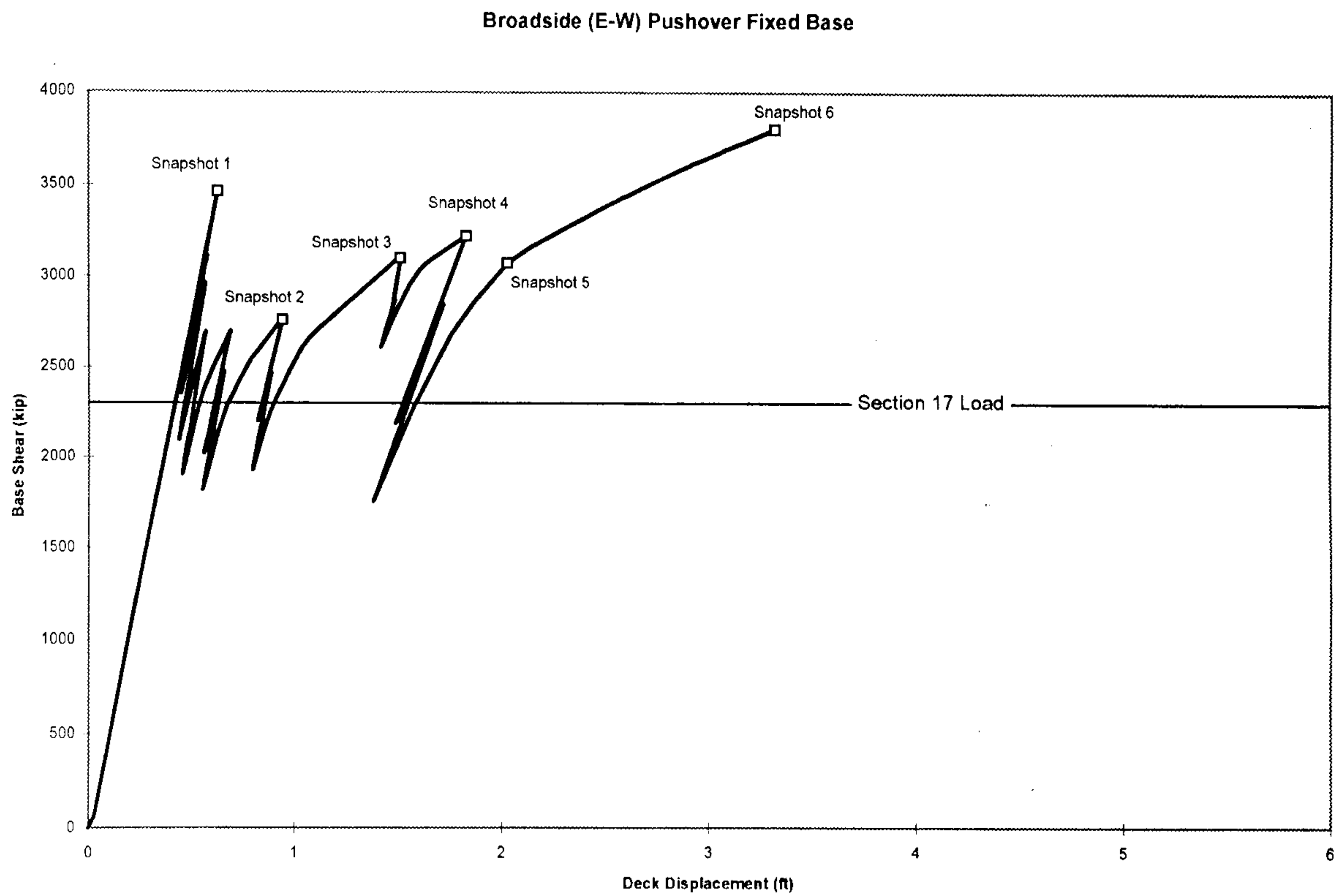
Project: Benchmark Model: 315 Version:Sat Aug 13 16:17:01 1994 *



CAP Broadside (E-W) Push - snapshot 5

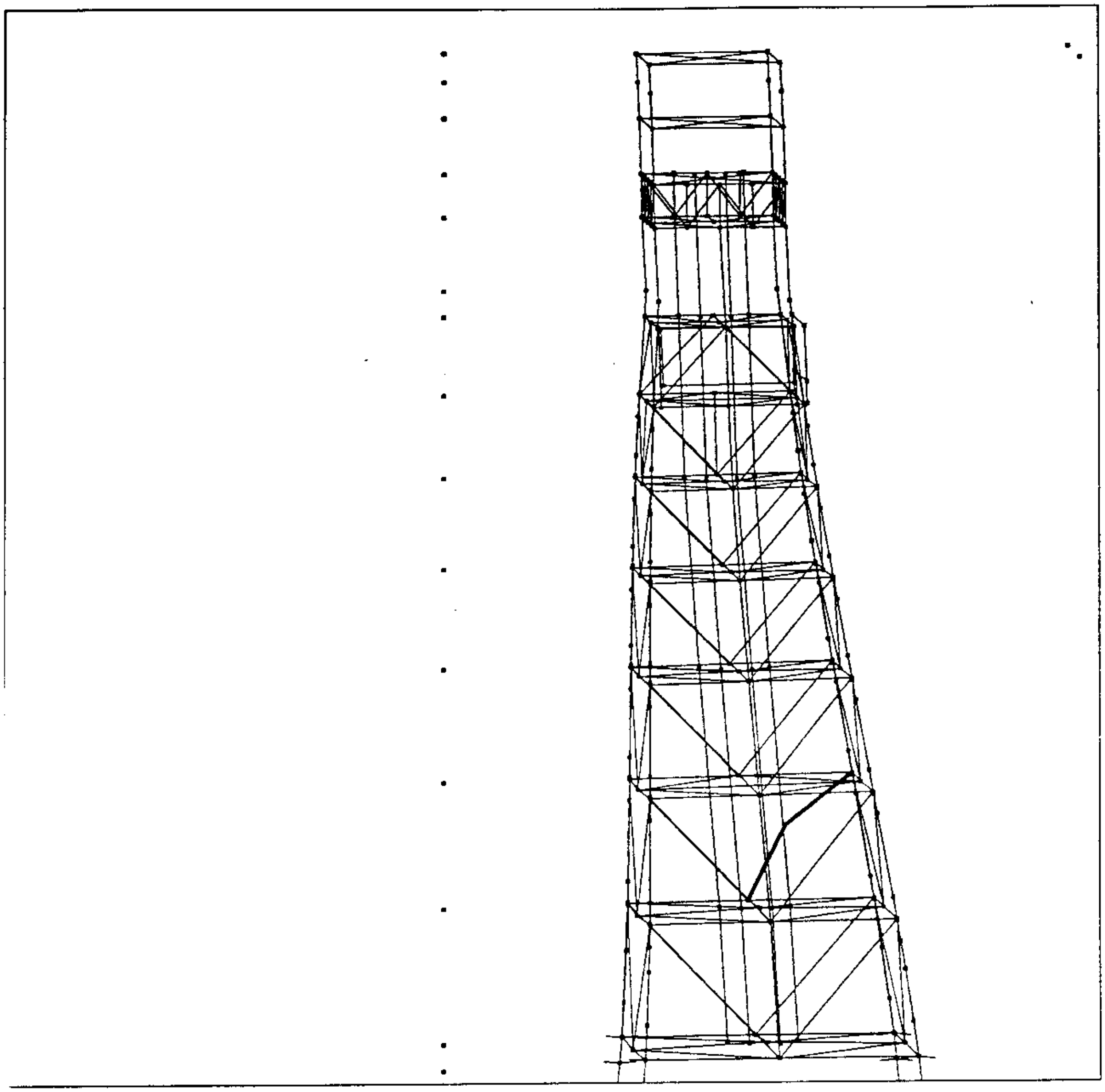
Inelastic Events Legend

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



Broadside (E-W) Pushover Fixed Base
Base Shear (kip)
4000
3500
3000
2500
2000
1500
1000
500
0
0
1
2
3
4
5
6
Deck Displacement (ft)
Snapshot 1
Snapshot 2
Snapshot 3
Snapshot 4
Snapshot 5
Snapshot 6
Section 17 Load

Project: Benchmark Model: 315-C-Fixed Version: 1Sat Aug 13 17:54:56 1994 *

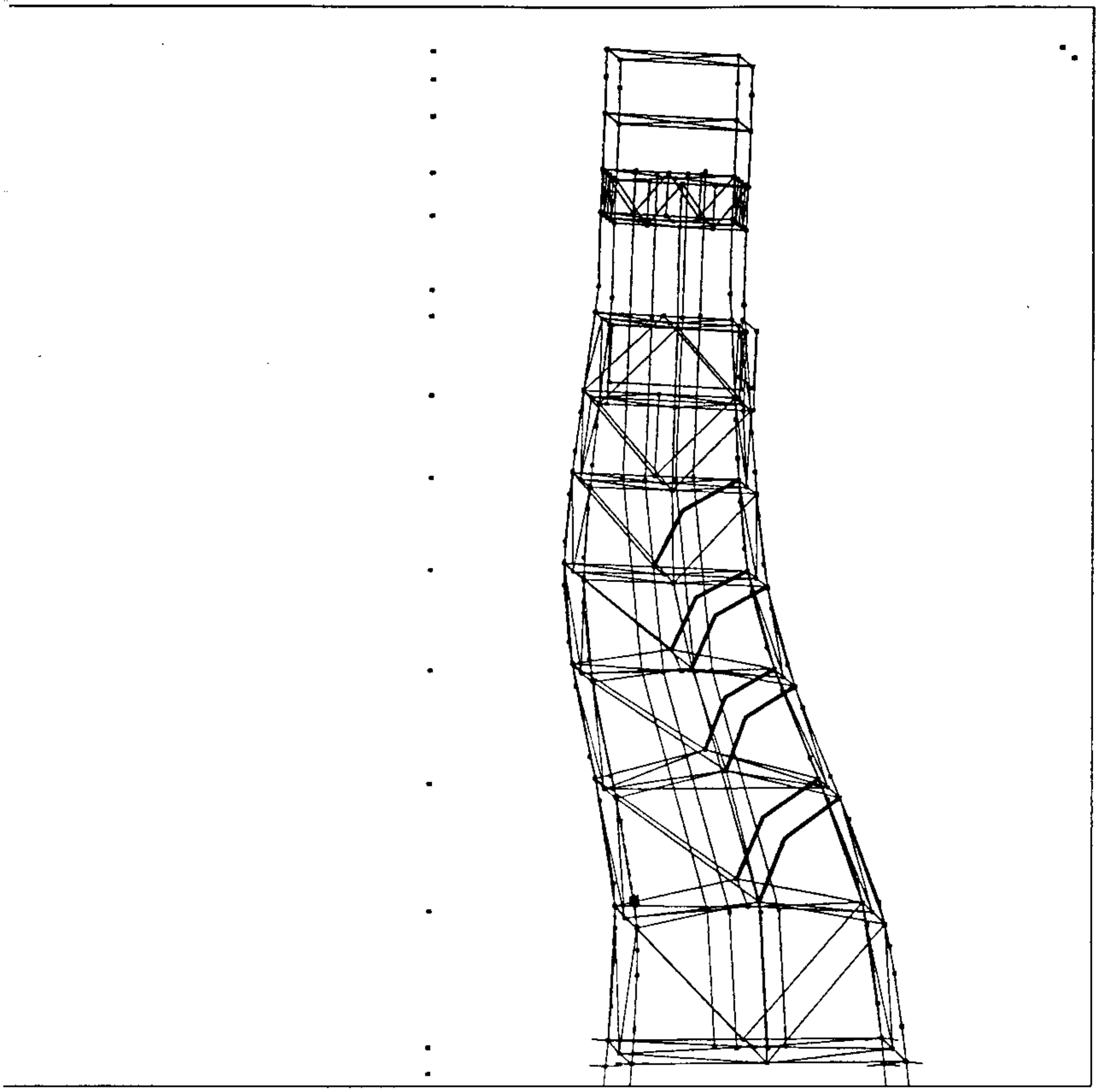


CAP Broadside (E-W) Push, Fixed Base - snapshot 1

Inelastic Events Legend

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

Project: Benchmark Model: 315-C-Fixed Version: 1Sat Aug 13 17:55:18 1994 *

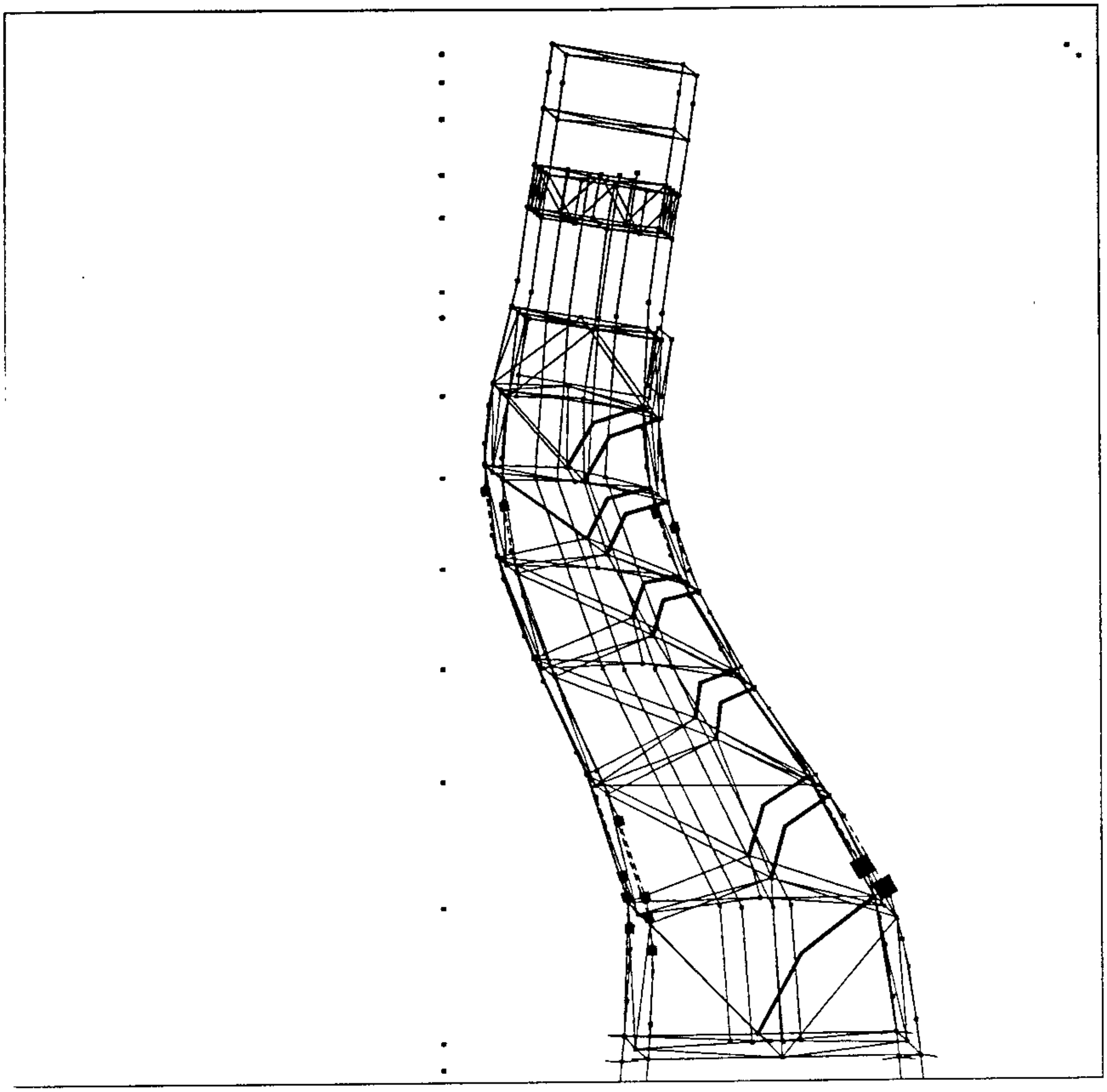


CAP Z X   Broadside (E-W) Push, Fixed Base - snapshot 2

Inelastic Events Legend

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

Project: Benchmark Model: 315-C-Fixed Version: 1Sat Aug 13 17:56:04 1994 *

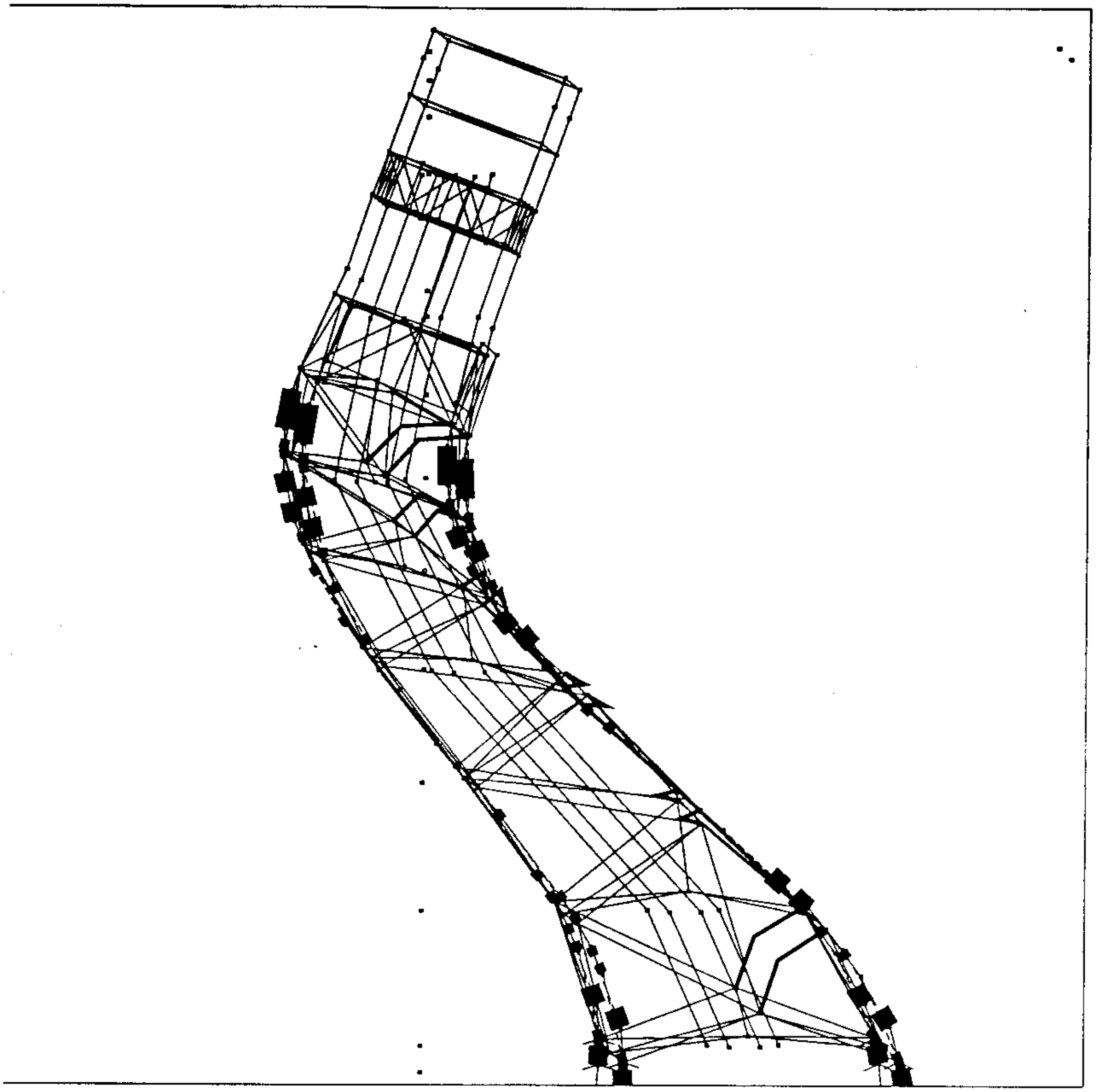


Broadside (E-W) Push, Fixed Base - snapshot 4

Inelastic Events Legend

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

Project: Benchmark Model: 315-C-Fixed Version: 1Sat Aug 13 17:56:44 1994 *



CAP Broadside (E-W) Push, Fixed Base - snapshot 6

Inelastic Events Legend

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

PARTICIPANT "D"

BENCHMARK ANALYSIS

TRIAL APPLICATION OF THE DRAFT API RP2A-WSD PROCEDURES FOR ASSESSMENT OF EXISTING PLATFORMS

JOINT INDUSTRY PROJECT

Summary

An ultimate strength analysis was performed on a 4 pile platform located in Ship Shoal Block 224 of the Gulf of Mexico. This platform was selected as a benchmark platform to be analyzed by several different organizations as part of a joint industry project sponsored by the MMS and managed by PMB Engineering, Inc. These analyses are to be compared to determine the variability of the ultimate strength results and to evaluate the use and application of the API draft guidelines on the assessment of existing platforms.

The results of this ultimate strength analysis showed that the benchmark platform passed the assessment process for the designated exposure category of a manned, evacuated platform with significant environmental impact. The minimum reserve strength ratio (for a diagonal approach) was 1.7 and the ultimate lateral load capacity was 2200 kips or 138% of the lateral load required by the ultimate strength assessment criteria for the Gulf of Mexico. The governing failure mechanism was the ultimate capacity of the pile foundation.

Part A: Benchmark Analysis

A.1 Environmental Criteria

Three wave approach directions were used with the following environmental criteria for each direction:

1. Approach angle with respect to Platform North = 0 degrees (315 deg. True)
   Wave height = 65.2 feet
   Wave period = 13.5 sec.
   Current Profile = 2.3 knots @ 275 deg. True
   Storm Surge = 3.5 feet
   Wind Speed = 85 knots @ 10 meters above msl
   Wave load in Deck = 110 kips based on 0 deg. approach with 65.2 foot wave

2. Approach angle with respect to Platform North = 270 degrees (225 deg. True)
   Wave height = 55.8 feet
   Wave period = 13.5 sec.
   Current Profile = 2.3 knots @ 275 deg. True
   Storm Surge = 3.5 feet
   Wind Speed = 85 knots @ 10 meters above msl
   Wave load in Deck = 38 kips based on 270 deg. approach with 55.8 foot wave

3. Approach angle with respect to Platform North = <u>315 degrees</u> (270 deg. True)
Wave height = <u>64.0 feet</u>
Wave period = <u>13.5 sec.</u>
Current Profile = <u>2.3 knots</u> @ 275 deg. True
Storm Surge = <u>3.5 feet</u>
Wind Speed = <u>85 knots</u> @ 10 meters above msl
Wave load in Deck = <u>98 kips</u> based on 315 deg. approach with 64.0 foot wave

The wave and current directions were based on API RP2A 20th edition with the wave heights for each direction calculated based on the factors given in RP2A figure 2.3.4-4. Because the waves generated forces on the deck, the platform wave and current loadings were calculated separately for the jacket and the deck. The loading on the jacket was calculated with a wave passing just below the bottom of steel of the +33 foot deck elevation. The wave loads in the deck were calculated with the procedure given in section C17.6 of the API RP2A draft section 17. The wave-induced horizontal fluid velocities were based on a 55 foot wave height and were not recalculated for each wave height. The total platform loading was calculated as the sum of the jacket and deck loadings.

The 20th edition RP2A recipe was used for calculating the wave and current loading below the +33 foot elevation, with the following criteria:

- Wave kinematics factor:	0.88
- Current blockage:	0.80 end-on/broadside
	0.85 diagonal
- Marine Growth:	1.5" (+1 ft. to mudline)
- Drag Coefficient:	0.65 (smooth)
	1.05 (rough)
- Inertia Coefficient:	1.60 (smooth)
	1.20 (rough)
- Wave theory:	Stokes 5th
- Conductor Shielding:	neglected
- Apparent wave period:	14 - 14.4 sec

A.1.1 API RP2A 20th ed. Environmental criteria

For use in the calculation of the platform reserve strength ratio, the lateral loads for the 100 year loading based on the RP2A 20th ed. criteria were determined. These criteria are listed below:

a. Wave height (max):	63 feet
b. Storm Tide:	3.5 feet
c. Deck min. height:	49.5 feet
d. Wave and Current direction:	Fig. 2.3.4-4 & 2.3.4-5
e. Current Speed:	2.1 knots
f. Wave Period:	13.0 sec.
g. Wind Speed:	80 knots

The associated wave height and current for various approach angles are as follows:

Approach Angle w.r.t Str. North/True North	Height (ft)	Current (kts)	Current Angle (True)
0 / 315	61.3	2.1	275
45 / 0	56.4	2.0	285
90 / 45	48.3	1.5	315
135 / 90	44.1	0.8	260
180 / 135	44.1	1.2	205
225 / 180	45.9	1.9	255
270 / 225	52.5	2.1	275
315 / 270	60.2	2.1	275
335 / 290	63.0	2.1	275

The lateral loads (kips) for each wave approach angle are listed below:

Approach Angle w.r.t. Str. North / True North	Jacket Force	Deck Force	Base Shear Total
0 / 315	1216	64	1280
45 / 0	1143	41	1184
90 / 45	1162	0	1162
135 / 90	900	0	900
180 / 135	913	0	913
225 / 180	1034	0	1034
270 / 225	1273	17	1290
315 / 270	1247	63	1310

A.2 3-D Model Generation

The structure model was developed based on the drawings and information given in the Design Basis Document including the 3 revisions issued. The model was generated with the SACS program by Engineering Dynamics, Inc. and converted for use in the nonlinear structural analysis program, Capacity Analysis Program (CAP) by PMB Engineering, Inc. The SACS program was used initially to perform a design level analysis for comparison with the ultimate strength analysis. The model (as shown in Figure A.2.1) included the jacket, deck, piles, conductors and appurtenances such as boat landings and risers.

The ultimate strength analysis utilized the following elements for member modeling:

Legs/Piles	Beam Column	(refer to CAP documentation)
Vertical Diagonal Braces	Marshal Strut	"
Horizontal Braces	Linear Beam	"
Deck Members	Linear Beam	"

The major structural joints were checked with the SACS program for the design level analysis. These joints were determined to have acceptable stress ratios for the design level loading and were therefore considered to be adequate to transfer the ultimate strength loads.

The soil springs were generated by the CAP program based on the parameters given in the Design Basis Document. The ultimate capacity of the piles was calculated by the CAP program and checked with hand calculations.

The material yield strength for all structural members (jacket, deck and foundation) was 42 ksi.

A.3 Software Description

The PMB Capacity Analysis Program (CAP) was used and is described in detail in the PMB documentation.

A.4 Ultimate Strength Analysis Results

The results of the ultimate strength analyses are summarized for each approach direction in the following Figures and Tables:

Figure A.4.1	0 degree approach ultimate lateral load vs. deck deflection.
Figure A.4.2	270 degree approach ultimate lateral load vs. deck deflection.
Figure A.4.3	315 degree approach ultimate lateral load vs. deck deflection.
Table A.4.1	0 degree approach tabulated results for load step, displacement and load.
Table A.4.2	270 degree approach tabulated results for load step, displacement and load.
Table A.4.3	315 degree approach tabulated results for load step, displacement and load.
Table A.4.4	315 degree approach tabulated failure modes and inelastic events.
Figures A.4.4	Governing pile capacity curve for Pile A2 for 315 degree approach angle.
Figures A.4.5	Governing pile capacity curve for Pile B1 for 315 degree approach angle.
Figure A.4.6	Deflected shape for 315 degree approach angle.

A.5 Design Level Analysis

A design level analysis is not applicable for this benchmark platform because of the inadequate deck height. However, a design level analysis was performed to compare the results with the ultimate strength analysis. The design level loading included deck forces generated by the waves hitting the 1st deck level (+33' elev.).

A.5.1 Environmental criteria

a. Wave height:	55 feet
b. Storm Tide:	3 feet
c. Deck min. height:	45.8 feet
d. Wave and Current direction:	Omni
e. Current Speed:	1.6 knots
f. Wave Period:	12.1 sec.
g. Wind Speed:	65 knots

The same procedure and recipe for calculating the environmental loading as discussed above in the ultimate strength analysis was used.

The lateral loads (kips) for each wave approach angle are listed below:

Approach Angle w.r.t Str. North	Approach Angle w.r.t. True North	Jacket Force	Deck Force	Base Shear Total
0	315	1161	26	1187
45	0	1156	28	1184
90	45	1230	26	1256 *
135	90	1159	28	1187 *
180	135	1161	26	1187 *
225	180	1157	28	1185 *
270	225	1230	26	1256
315	270	1167	28	1195

The applied lateral loading for the approach angles marked with a * by the base shear values were reduced by load factors to not exceed the RP2A 20th edition lateral loading.

A.5.2 Design Level Results

The design level analysis results showed that the platform legs, braces and pile foundation were not overstressed with the applied loading and the platform passed the design level assessment. The governing component is the pile foundation with a reaction of 2026 kips and an ultimate capacity (compression) of 3390 kips, resulting in a factor of safety of 1.67 (1.5 required). These results are similar to the ultimate strength results which also showed the governing failure component to be the pile foundation.

FIGURE A.4.1 — 0 DEGREE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1410 kips

100 year, 20th ed. ref. load (Sref) = 1280 kips

Ultimate Strength Analysis Load (Suso) = 1440 kips

Ultimate Capacity (Ru) = 2764 kips

Reserve Strength Ratio = 2.1

Platform Failure Mode = Foundation

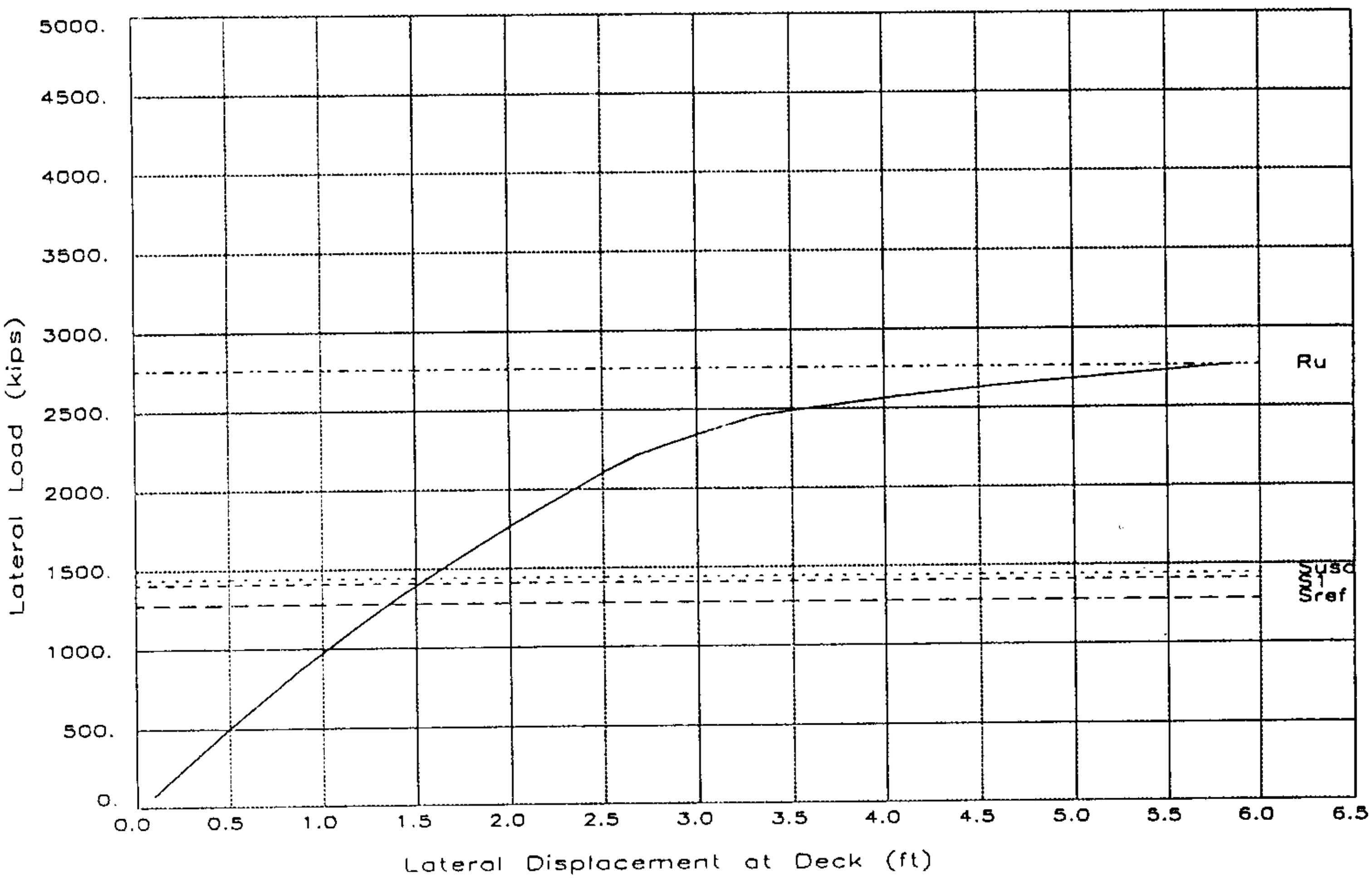


FIGURE A.4.2 – 270 DEGREE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1375 kips

100 year, 20th ed. ref. load (Sref) = 1290 kips

Ultimate Strength Analysis Load (Susa) = 1390 kips

Ultimate Capacity (Ru) = 2623 kips

Reserve Strength Ratio = 2.0

Platform Failure Mode = Foundation

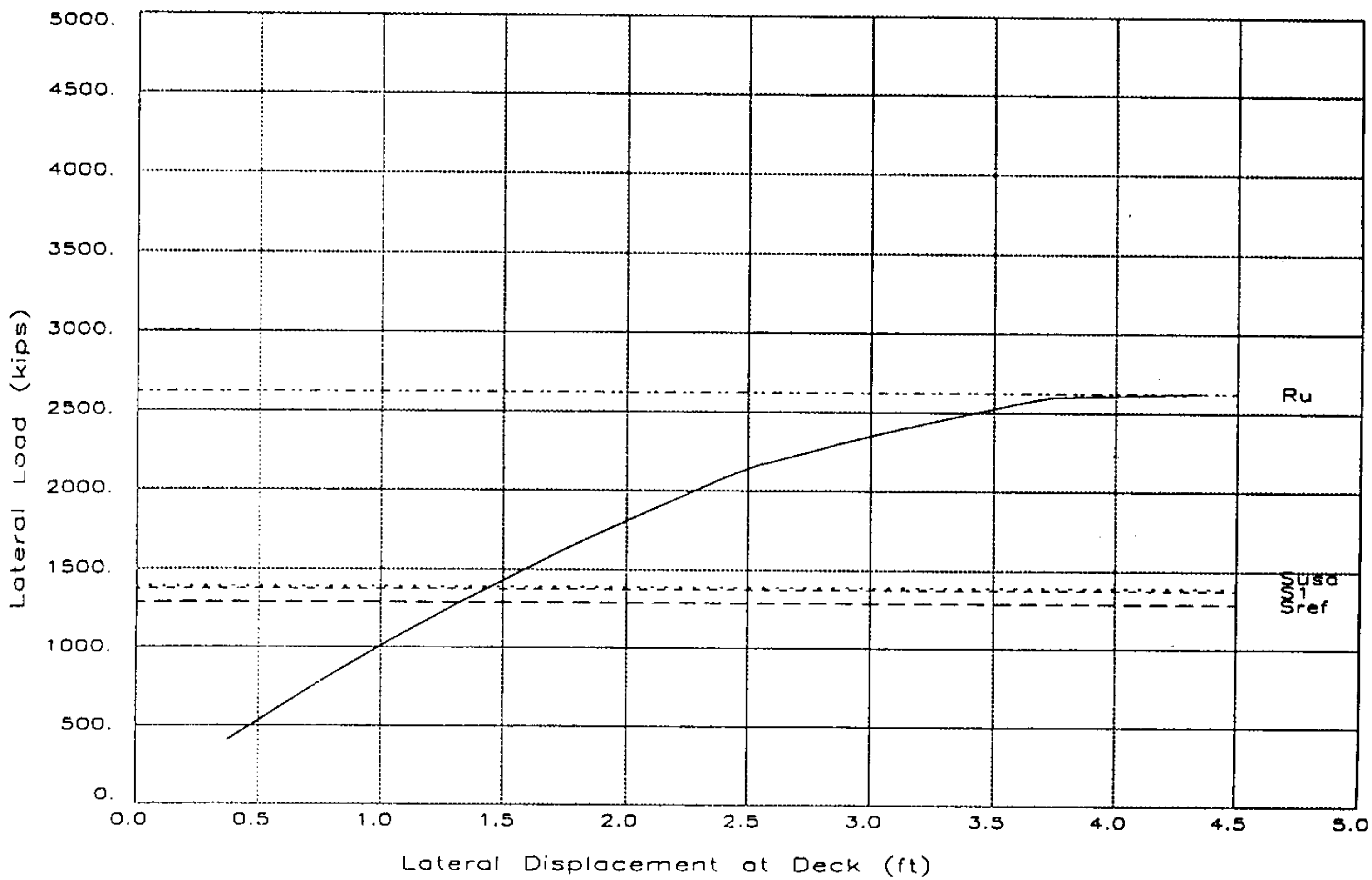


FIGURE A.4.3 — 315 DEGREE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1500 kips

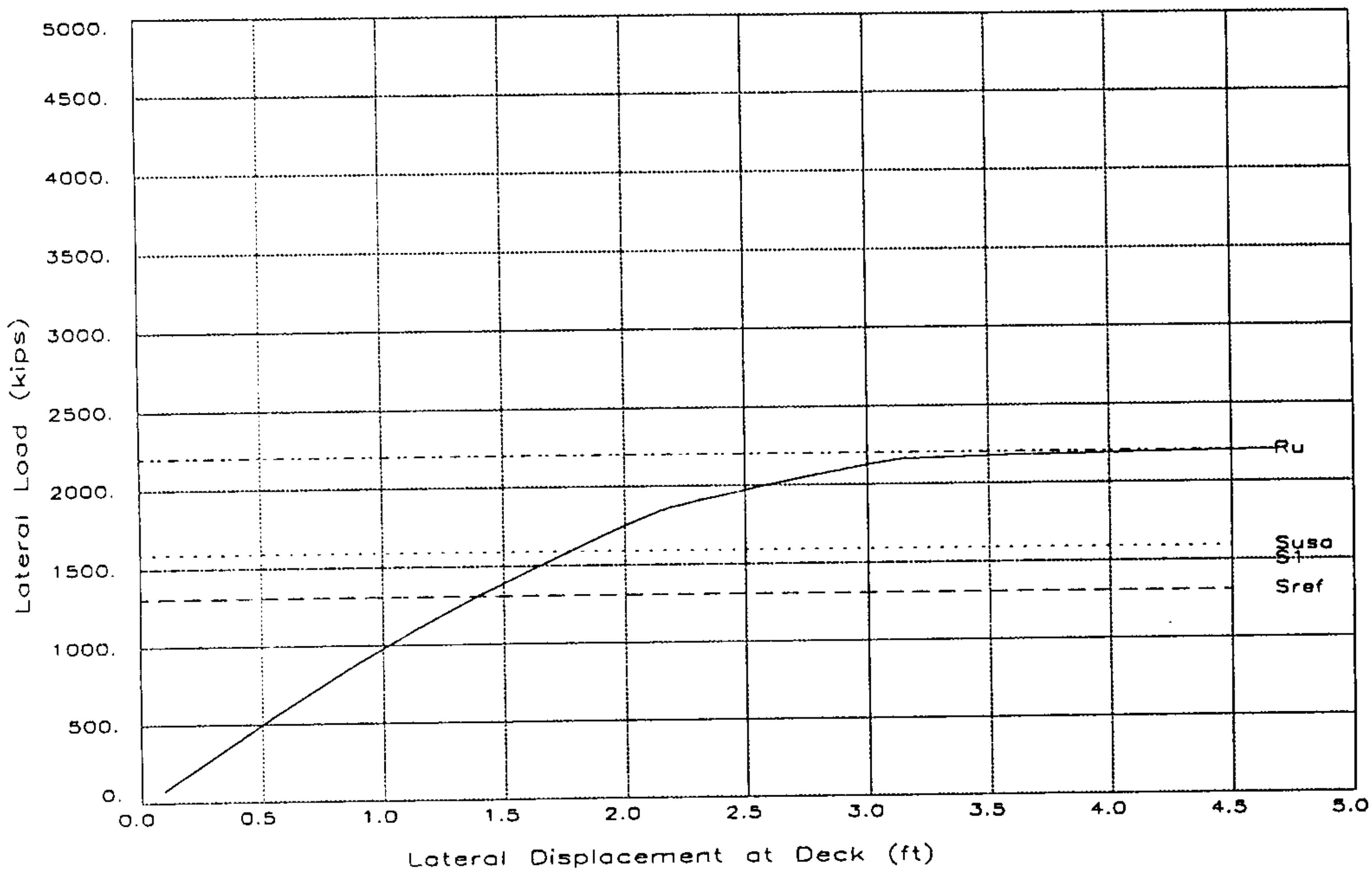
100 year, 20th ed. ref. load (Sref) = 1310 kips

Ultimate Strength Analysis Load (Susa) = 1590 kips

Ultimate Capacity (Ru) = 2200 kips

Reserve Strength Ratio = 1.7

Platform Failure Mode = Foundation



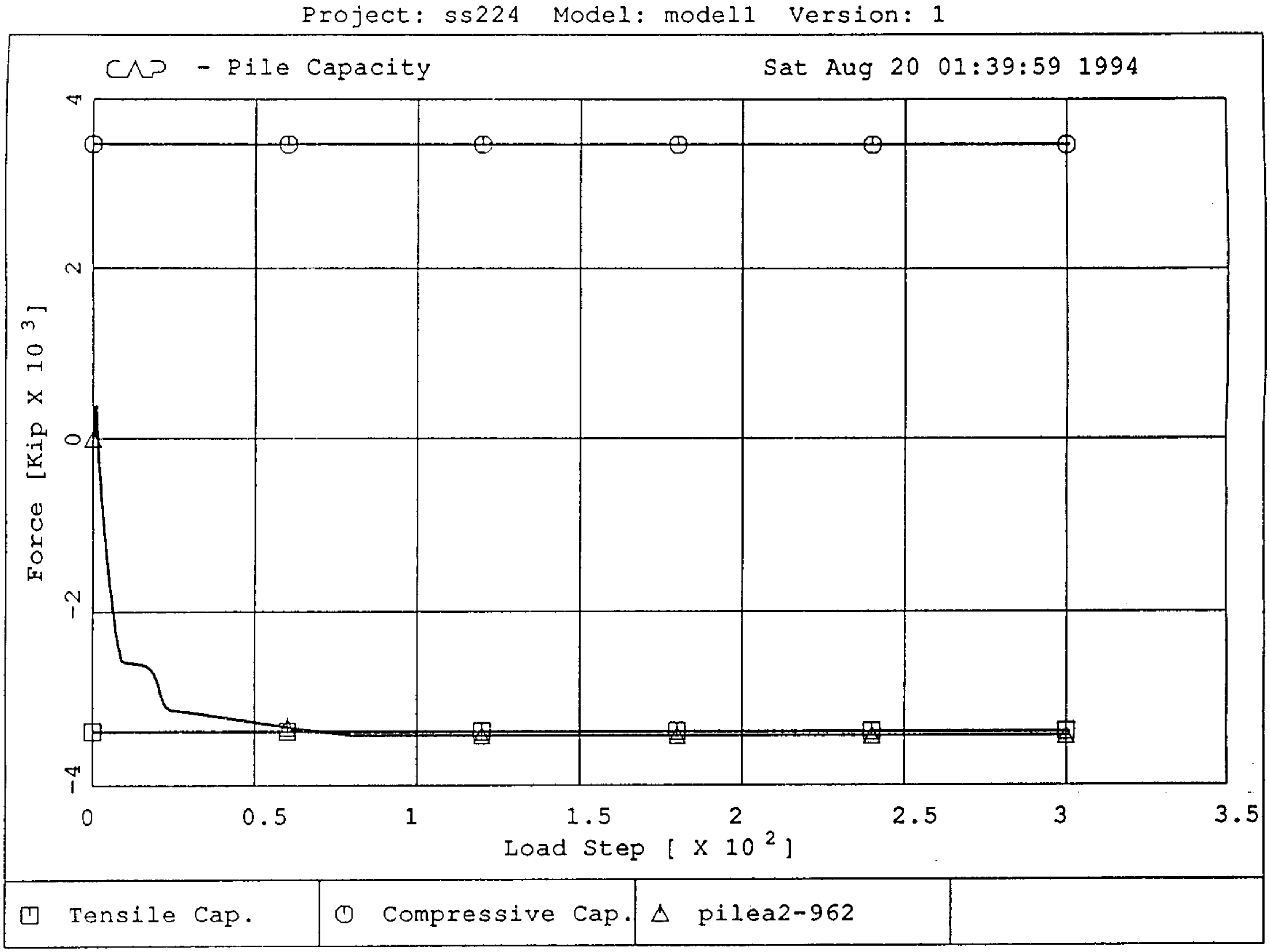


Figure A.4.4 135 deg. pile cap.

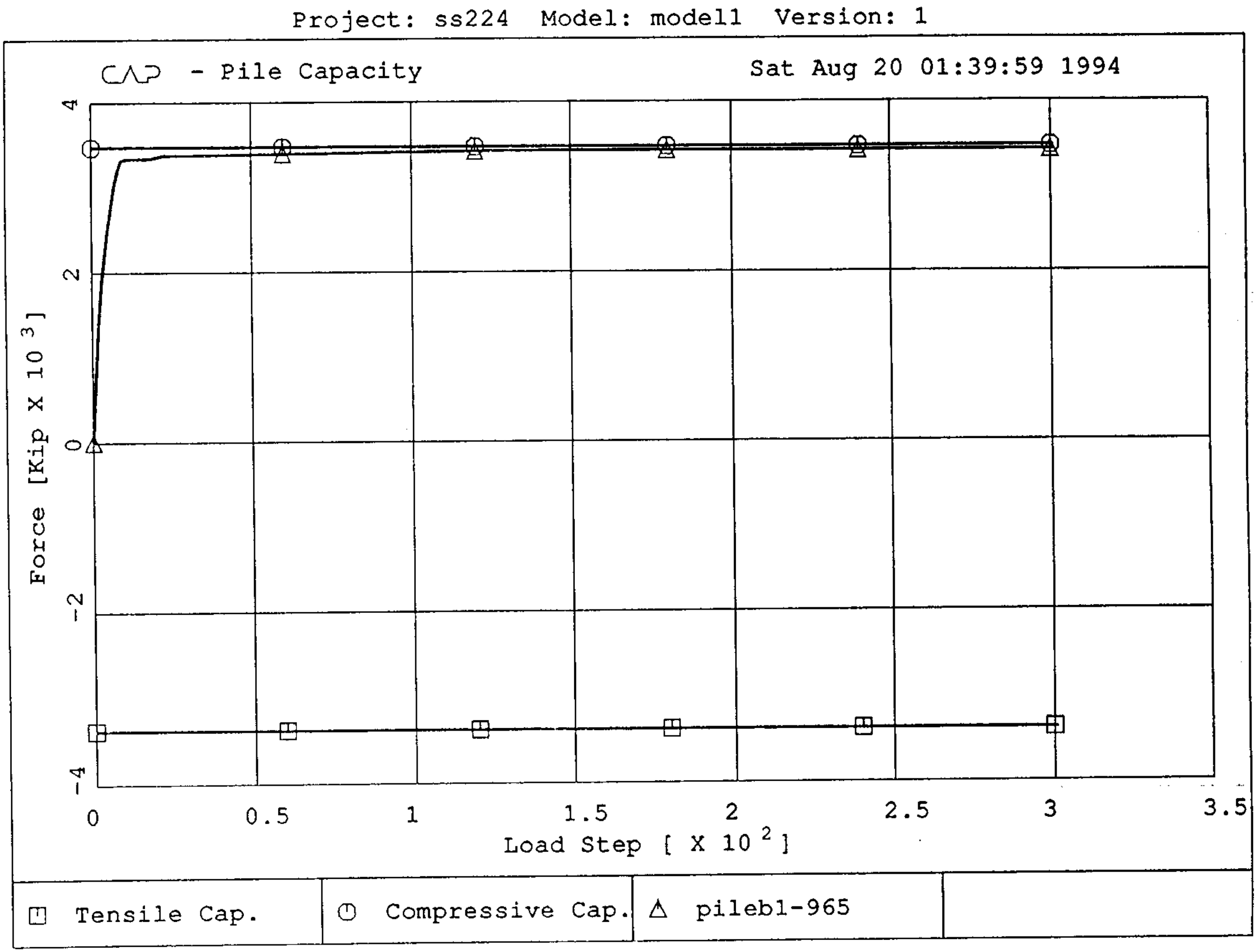
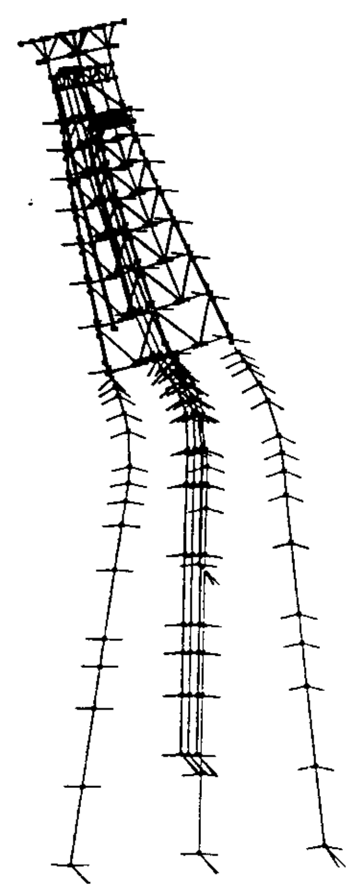


Figure A.4.5 135 deg. pile cap.

Project: ss224 Model: model1 Version: 1 Sat Aug 20 09:59:24 1994 *



CAP z x Figure A.4.6 Deflected Shape at 315 deg.

** TABLE A.4.1 0 DEGREE DIRECTION

CAP Results Table Sun Aug 21 01:14:30 1994

Project: ss224 Model: model1 Version: 1

Cut Plane Force Fx Kip

0 degree direction

Load Step	X Deck Disp Ft	Force Kips
1	0.090	69.778
2	0.490	493.597
3	0.886	873.763
4	1.187	1138.643
5	1.419	1333.144
6	1.616	1487.128
7	1.858	1666.409
8	2.045	1797.934
9	2.191	1895.538
10	2.370	2014.628
11	2.447	2065.929
12	2.524	2116.805
13	2.602	2163.784
14	2.645	2188.502
15	2.664	2199.138
16	2.696	2217.536
17	2.699	2219.121
18	2.701	2219.877
19	2.705	2221.273
20	2.711	2223.689
21	2.721	2227.875
22	2.740	2235.125
23	2.771	2247.682
24	2.826	2269.431
25	2.921	2306.752
26	3.037	2351.218
27	3.239	2425.838
28	3.266	2435.104
29	3.313	2451.167
30	3.333	2458.000
31	3.335	2458.739
32	3.339	2459.854
33	3.341	2460.203
34	3.346	2460.907
35	3.353	2462.126
36	3.366	2464.230
37	3.389	2467.877
38	3.429	2474.127
39	3.497	2484.652
40	3.616	2503.509
41	3.760	2526.402
42	4.012	2564.079
43	4.075	2572.430
44	4.102	2575.992
45	4.114	2577.534
46	4.134	2580.234
47	4.143	2581.381
48	4.147	2581.878
49	4.154	2582.739
50	4.160	2583.641
51	4.163	2584.021
52	4.168	2584.656
53	4.175	2585.496
54	4.187	2587.062
55	4.192	2587.731
56	4.201	2588.901
57	4.205	2589.401
58	4.211	2590.271
59	4.218	2591.174
60	4.221	2591.560
61	4.226	2592.227
62	4.233	2593.107
63	4.240	2593.976
64	4.247	2594.865
65	4.250	2595.250
66	4.255	2595.917
67	4.262	2596.801
68	4.269	2597.684
69	4.275	2598.571
70	4.282	2599.455
71	4.289	2600.338
72	4.296	2601.221
73	4.303	2602.103
74	4.309	2602.986
75	4.316	2603.868
76	4.323	2604.751
77	4.330	2605.633
78	4.337	2606.515
79	4.343	2607.397
80	4.350	2608.279
81	4.357	2609.161
82	4.364	2610.043
83	4.371	2610.922
84	4.378	2611.802
85	4.384	2612.680
86	4.391	2613.560
87	4.398	2614.438
88	4.405	2615.313
89	4.412	2616.186
90	4.418	2617.056
91	4.425	2617.926
92	4.432	2618.795
93	4.439	2619.663
94	4.445	2620.532
95	4.452	2621.401
96	4.459	2622.269
97	4.466	2623.137
98	4.473	2624.007
99	4.479	2624.870
100	4.486	2625.733
101	4.493	2626.592
102	4.500	2627.458
103	4.506	2628.325
104	4.513	2629.197
105	4.520	2630.052
106	4.527	2630.905
107	4.533	2631.756
108	4.540	2632.612
109	4.547	2633.480
110	4.554	2634.335
111	4.561	2635.200
112	4.567	2636.050
113	4.574	2636.890
114	4.581	2637.751
115	4.588	2638.623
116	4.594	2639.441
117	4.601	2640.154
118	4.608	2640.860
119	4.615	2641.558

120	4.622	2642.254
121	4.629	2642.948
122	4.635	2643.642
123	4.642	2644.334
124	4.649	2645.026
125	4.656	2645.718
126	4.663	2646.410
127	4.670	2647.102
128	4.676	2647.794
129	4.683	2648.486
130	4.690	2649.178
131	4.697	2649.869
132	4.704	2650.562
133	4.710	2651.253
134	4.717	2651.942
135	4.724	2652.630
136	4.731	2653.318
137	4.738	2654.006
138	4.745	2654.695
139	4.751	2655.383
140	4.758	2656.071
141	4.765	2656.759
142	4.772	2657.447
143	4.779	2658.135
144	4.786	2658.823
145	4.792	2659.511
146	4.799	2660.199
147	4.806	2660.888
148	4.813	2661.576
149	4.820	2662.264
150	4.826	2662.952
151	4.833	2663.640
152	4.840	2664.328
153	4.847	2665.016
154	4.854	2665.705
155	4.861	2666.393
156	4.867	2667.081
157	4.874	2667.769
158	4.881	2668.453
159	4.888	2669.135
160	4.895	2669.818
161	4.902	2670.500
162	4.908	2671.182
163	4.915	2671.865
164	4.922	2672.547
165	4.929	2673.229
166	4.936	2673.912
167	4.943	2674.594
168	4.949	2675.276
169	4.956	2675.959
170	4.963	2676.641
171	4.970	2677.323
172	4.977	2678.006
173	4.984	2678.688
174	4.990	2679.370
175	4.997	2680.052
176	5.004	2680.735
177	5.011	2681.417
178	5.018	2682.099
179	5.024	2682.782
180	5.031	2683.463
181	5.038	2684.145
182	5.045	2684.826
183	5.052	2685.508
184	5.059	2686.189
185	5.065	2686.871
186	5.072	2687.553
187	5.079	2688.234
188	5.086	2688.916
189	5.093	2689.597
190	5.100	2690.279
191	5.106	2690.960
192	5.113	2691.642
193	5.120	2692.324
194	5.127	2693.005
195	5.134	2693.687
196	5.141	2694.368
197	5.147	2695.050
198	5.154	2695.731
199	5.161	2696.413
200	5.168	2697.095
201	5.175	2697.776
202	5.182	2698.458
203	5.188	2699.139
204	5.195	2699.821
205	5.202	2700.502
206	5.209	2701.184
207	5.216	2701.865
208	5.223	2702.546
209	5.229	2703.228
210	5.236	2703.910
211	5.243	2704.591
212	5.250	2705.272
213	5.257	2705.951
214	5.263	2706.631
215	5.270	2707.311
216	5.277	2707.991
217	5.284	2708.671
218	5.291	2709.349
219	5.298	2710.028
220	5.304	2710.706
221	5.311	2711.384
222	5.318	2712.063
223	5.325	2712.741
224	5.332	2713.419
225	5.339	2714.098
226	5.345	2714.776
227	5.352	2715.454
228	5.359	2716.133
229	5.366	2716.811
230	5.373	2717.489
231	5.379	2718.167
232	5.386	2718.845
233	5.393	2719.524
234	5.400	2720.202
235	5.407	2720.880
236	5.414	2721.558
237	5.420	2722.236
238	5.427	2722.914
239	5.434	2723.592
240	5.441	2724.269
241	5.448	2724.947
242	5.454	2725.625
243	5.461	2726.303
244	5.468	2726.978
245	5.475	2727.649
246	5.482	2728.320
247	5.489	2728.992
248	5.495	2729.663
249	5.502	2730.335
250	5.509	2731.006
251	5.516	2731.677

252	5.523	2732.346
253	5.529	2733.010
254	5.536	2733.674
255	5.543	2734.338
256	5.550	2735.002
257	5.557	2735.666
258	5.564	2736.330
259	5.570	2736.994
260	5.577	2737.657
261	5.584	2738.321
262	5.591	2738.985
263	5.598	2739.649
264	5.604	2740.313
265	5.611	2740.974
266	5.618	2741.622
267	5.625	2742.273
268	5.632	2742.924
269	5.638	2743.576
270	5.645	2744.227
271	5.652	2744.878
272	5.659	2745.530
273	5.666	2746.181
274	5.672	2746.832
275	5.679	2747.483
276	5.686	2748.135
277	5.693	2748.786
278	5.700	2749.438
279	5.707	2750.088
280	5.713	2750.740
281	5.720	2751.390
282	5.727	2752.042
283	5.734	2752.692
284	5.741	2753.344
285	5.747	2753.994
286	5.754	2754.646
287	5.761	2755.296
288	5.768	2755.948
289	5.775	2756.599
290	5.781	2757.250
291	5.788	2757.901
292	5.795	2758.551
293	5.802	2759.202
294	5.809	2759.853
295	5.816	2760.503
296	5.822	2761.154
297	5.829	2761.804
298	5.836	2762.454
299	5.843	2763.102
300	5.850	2763.748
301	5.856	2764.394

*** TABLE A.4.2 ***

CAP Results Table — Mon Aug 22 04:44:43 1994

Project: ss224 Model: model1 Version: 1

Cut Plane Force Fy - Kip

270 deg. direction

Load Step	Y Deck Disp Ft	Force Kips
2	0.375	404.204
3	0.746	777.063
4	1.029	1036.981
5	1.309	1280.208
6	1.550	1471.852
7	1.717	1599.117
8	1.862	1707.274
9	2.040	1832.817
10	2.218	1956.301
11	2.397	2076.448
12	2.514	2148.889
13	2.523	2153.627
14	2.528	2155.584
15	2.536	2159.071
16	2.550	2165.111
17	2.575	2175.565
18	2.617	2193.606
19	2.691	2223.755
20	2.754	2249.710
21	2.864	2294.373
22	3.055	2368.972
23	3.089	2381.284
24	3.147	2402.595
25	3.153	2404.899
26	3.164	2408.883
27	3.169	2410.545
28	3.174	2412.518
29	3.180	2414.124
30	3.185	2415.547
31	3.190	2417.372
32	3.196	2419.332
33	3.205	2422.859
34	3.209	2424.376
35	3.211	2425.029
36	3.214	2426.139
37	3.220	2428.150
38	3.222	2429.028
39	3.226	2430.550
40	3.228	2431.205
41	3.231	2432.324
42	3.237	2434.237
43	3.242	2436.119
44	3.248	2438.121
45	3.253	2440.118
46	3.259	2442.090
47	3.264	2444.036
48	3.270	2445.959
49	3.275	2447.871
50	3.281	2449.775
51	3.286	2451.676
52	3.292	2453.573
53	3.297	2455.469
54	3.303	2457.366
55	3.308	2459.262
56	3.313	2461.158
57	3.319	2463.053
58	3.324	2464.945
59	3.330	2466.833
60	3.335	2468.722
61	3.341	2470.608
62	3.346	2472.489
63	3.352	2474.370
64	3.357	2476.252
65	3.363	2478.126
66	3.368	2479.999
67	3.373	2481.872
68	3.379	2483.745
69	3.384	2485.617
70	3.390	2487.490
71	3.395	2489.361
72	3.401	2491.234
73	3.406	2493.106
74	3.412	2494.978
75	3.417	2496.850
76	3.423	2498.721
77	3.428	2500.593
78	3.433	2502.465
79	3.439	2504.336
80	3.444	2506.208
81	3.450	2508.079
82	3.455	2509.944
83	3.461	2511.805
84	3.466	2513.667
85	3.472	2515.527
86	3.477	2517.388
87	3.482	2519.248
88	3.488	2521.109
89	3.493	2522.969
90	3.499	2524.830
91	3.504	2526.690
92	3.510	2528.551
93	3.515	2530.411
94	3.521	2532.272
95	3.526	2534.132
96	3.531	2535.992
97	3.537	2537.858
98	3.542	2539.704
99	3.548	2541.529
100	3.553	2543.344
101	3.558	2545.104
102	3.564	2546.852
103	3.569	2548.606
104	3.574	2550.420
105	3.580	2552.265
106	3.585	2554.096
107	3.591	2555.889
108	3.596	2557.611
109	3.601	2559.127
110	3.607	2560.676
111	3.612	2562.222
112	3.617	2563.769
113	3.623	2565.314
114	3.628	2566.860
115	3.633	2568.402
116	3.638	2569.942
117	3.644	2571.483
118	3.649	2573.024
119	3.654	2574.564
120	3.660	2576.104

121	3.665	2577.645
122	3.670	2579.186
123	3.676	2580.726
124	3.681	2582.267
125	3.686	2583.784
126	3.692	2585.250
127	3.697	2586.734
128	3.702	2588.219
129	3.708	2589.704
130	3.713	2591.189
131	3.719	2592.673
132	3.724	2594.158
133	3.730	2595.642
134	3.735	2597.042
135	3.740	2597.212
136	3.745	2597.559
137	3.750	2597.889
138	3.755	2598.221
139	3.760	2598.554
140	3.764	2598.887
141	3.769	2599.181
142	3.774	2599.381
143	3.779	2599.581
144	3.784	2599.780
145	3.786	2599.867
146	3.791	2600.065
147	3.796	2600.262
148	3.816	2601.057
149	3.824	2601.404
150	3.828	2601.555
151	3.829	2601.620
152	3.834	2601.817
153	3.836	2601.904
154	3.840	2602.054
155	3.845	2602.250
156	3.847	2602.338
157	3.851	2602.487
158	3.856	2602.684
159	3.858	2602.771
160	3.862	2602.921
161	3.866	2603.117
162	3.869	2603.205
163	3.874	2603.401
164	3.876	2603.489
165	3.879	2603.638
166	3.884	2603.834
167	3.886	2603.922
168	3.891	2604.118
169	3.893	2604.206
170	3.897	2604.355
171	3.902	2604.551
172	3.904	2604.639
173	3.909	2604.835
174	3.911	2604.923
175	3.915	2605.072
176	3.920	2605.268
177	3.922	2605.355
178	3.927	2605.553
179	3.932	2605.750
180	3.934	2605.837
181	3.937	2605.985
182	3.939	2606.051
183	3.942	2606.162
184	3.947	2606.358
185	3.949	2606.445
186	3.954	2606.641
187	3.956	2606.728
188	3.961	2606.923
189	3.966	2607.120
190	3.971	2607.318
191	3.975	2607.515
192	3.980	2607.712
193	3.985	2607.910
194	3.990	2608.106
195	3.999	2608.449
196	4.002	2608.600
197	4.009	2608.857
198	4.011	2608.971
199	4.016	2609.165
200	4.021	2609.360
201	4.023	2609.447
202	4.028	2609.642
203	4.030	2609.728
204	4.034	2609.876
205	4.039	2610.070
206	4.041	2610.156
207	4.046	2610.351
208	4.048	2610.437
209	4.052	2610.585
210	4.057	2610.779
211	4.059	2610.865
212	4.064	2611.060
213	4.066	2611.146
214	4.069	2611.294
215	4.074	2611.488
216	4.076	2611.575
217	4.081	2611.769
218	4.083	2611.855
219	4.087	2612.003
220	4.092	2612.197
221	4.094	2612.283
222	4.099	2612.477
223	4.101	2612.563
224	4.106	2612.758
225	4.111	2612.952
226	4.113	2613.038
227	4.117	2613.185
228	4.118	2613.250
229	4.121	2613.360
230	4.126	2613.552
231	4.131	2613.745
232	4.133	2613.832
233	4.138	2614.026
234	4.140	2614.112
235	4.143	2614.260
236	4.148	2614.453
237	4.150	2614.540
238	4.155	2614.734
239	4.158	2614.820
240	4.161	2614.968
241	4.166	2615.161
242	4.168	2615.248
243	4.173	2615.442
244	4.175	2615.528
245	4.179	2615.676
246	4.184	2615.869
247	4.186	2615.956
248	4.191	2616.149
249	4.193	2616.236
250	4.198	2616.430
251	4.203	2616.624
252	4.205	2616.710

253	4.208	2616.857
254	4.210	2616.922
255	4.213	2617.032
256	4.218	2617.226
257	4.220	2617.312
258	4.225	2617.506
259	4.227	2617.592
260	4.231	2617.739
261	4.235	2617.933
262	4.238	2618.019
263	4.242	2618.213
264	4.245	2618.299
265	4.249	2618.494
266	4.254	2618.687
267	4.256	2618.772
268	4.260	2618.918
269	4.262	2618.983
270	4.267	2619.176
271	4.269	2619.261
272	4.274	2619.454
273	4.276	2619.540
274	4.281	2619.733
275	4.283	2619.819
276	4.288	2620.013
277	4.293	2620.206
278	4.295	2620.292
279	4.298	2620.438
280	4.300	2620.502
281	4.303	2620.612
282	4.308	2620.804
283	4.310	2620.891
284	4.315	2621.083
285	4.317	2621.169
286	4.320	2621.316
287	4.325	2621.508
288	4.327	2621.594
289	4.332	2621.787
290	4.334	2621.873
291	4.338	2622.020
292	4.343	2622.212
293	4.345	2622.298
294	4.350	2622.491
295	4.352	2622.577
296	4.356	2622.724
297	4.361	2622.916
298	4.363	2623.002
299	4.368	2623.196
300	4.373	2623.389
301	4.375	2623.475

** TABLE A.4.3 315 DEG. APPROACH ***

CAP Results Table   Sat Aug 20 01:39:59 1994

Project: ss224 Model: model1 Version: 1

Cut Plane Force Fx - Kip

315 deg. direction

Load Step	X Deck Disp Ft	Force Kips
1	0.090	72.338
2	0.534	533.353
3	0.873	865.383
4	1.133	1097.983
5	1.394	1310.372
6	1.596	1462.651
7	1.844	1639.369
8	2.006	1750.900
9	2.148	1839.469
10	2.166	1850.084
11	2.174	1854.683
12	2.178	1856.656
13	2.184	1859.376
14	2.187	1860.549
15	2.193	1862.651
16	2.204	1866.292
17	2.222	1872.600
18	2.253	1883.524
19	2.306	1902.445
20	2.400	1935.116
21	2.561	1989.089
22	2.676	2025.554
23	2.726	2041.305
24	2.747	2048.096
25	2.757	2051.036
26	2.761	2052.298
27	2.768	2054.499
28	2.771	2055.446
29	2.776	2057.052
30	2.783	2059.266
31	2.790	2061.397
32	2.797	2063.511
33	2.805	2065.613
34	2.812	2067.712
35	2.819	2069.809
36	2.826	2071.905
37	2.833	2074.000
38	2.840	2076.094
39	2.847	2078.189
40	2.854	2080.283
41	2.862	2082.378
42	2.869	2084.473
43	2.876	2086.567
44	2.883	2088.662
45	2.890	2090.757
46	2.897	2092.851
47	2.904	2094.946
48	2.911	2097.040
49	2.918	2099.135
50	2.926	2101.229
51	2.933	2103.323
52	2.940	2105.415
53	2.947	2107.507
54	2.954	2109.597
55	2.961	2111.687
56	2.968	2113.776
57	2.975	2115.866
58	2.983	2117.955
59	2.990	2120.045
60	2.997	2122.134
61	3.004	2124.223
62	3.011	2126.252
63	3.018	2127.972
64	3.026	2129.730
65	3.033	2131.488
66	3.040	2133.246
67	3.047	2135.004
68	3.054	2136.761
69	3.062	2138.517
70	3.069	2140.272
71	3.076	2142.027
72	3.083	2143.782
73	3.091	2145.538
74	3.098	2147.293
75	3.105	2149.049
76	3.112	2150.804
77	3.120	2152.559
78	3.127	2154.312
79	3.134	2156.065
80	3.141	2157.795
81	3.149	2158.118
82	3.157	2158.435
83	3.164	2158.752
84	3.172	2159.077
85	3.185	2159.633
86	3.209	2160.596
87	3.211	2160.679
88	3.215	2160.859
89	3.223	2161.164
90	3.231	2161.481
91	3.238	2161.799
92	3.248	2162.197
93	3.264	2162.878
94	3.293	2164.057
95	3.296	2164.159
96	3.301	2164.379
97	3.303	2164.471
98	3.307	2164.637
99	3.314	2164.917
100	3.322	2165.242
101	3.325	2165.376
102	3.331	2165.616
103	3.333	2165.716
104	3.338	2165.896
105	3.345	2166.201
106	3.353	2166.526
107	3.356	2166.659
108	3.362	2166.899
109	3.365	2166.999
110	3.369	2167.180
111	3.377	2167.484
112	3.384	2167.809
113	3.388	2167.942
114	3.393	2168.182
115	3.396	2168.282
116	3.400	2168.463
117	3.408	2168.767
118	3.415	2169.092
119	3.419	2169.225

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120	3.425	2169.466
121	3.427	2169.565
122	3.431	2169.746
123	3.439	2170.051
124	3.447	2170.375
125	3.450	2170.508
126	3.456	2170.749
127	3.458	2170.849
128	3.463	2171.029
129	3.470	2171.333
130	3.478	2171.658
131	3.481	2171.791
132	3.487	2172.032
133	3.489	2172.132
134	3.494	2172.312
135	3.501	2172.617
136	3.509	2172.941
137	3.512	2173.074
138	3.518	2173.313
139	3.521	2173.413
140	3.525	2173.592
141	3.532	2173.896
142	3.540	2174.219
143	3.544	2174.352
144	3.549	2174.592
145	3.552	2174.691
146	3.556	2174.871
147	3.564	2175.121
148	3.595	2175.880
149	3.648	2177.164
150	3.672	2177.680
151	3.682	2177.904
152	3.699	2178.321
153	3.707	2178.488
154	3.720	2178.801
155	3.725	2178.927
156	3.735	2179.161
157	3.739	2179.255
158	3.747	2179.431
159	3.759	2179.736
160	3.765	2179.858
161	3.775	2180.087
162	3.779	2180.179
163	3.786	2180.350
164	3.798	2180.647
165	3.804	2180.766
166	3.813	2180.989
167	3.817	2181.078
168	3.824	2181.245
169	3.836	2181.534
170	3.841	2181.650
171	3.850	2181.867
172	3.866	2182.242
173	3.873	2182.393
174	3.876	2182.458
175	3.881	2182.580
176	3.890	2182.791
177	3.905	2183.156
178	3.912	2183.303
179	3.923	2183.577
180	3.928	2183.687
181	3.937	2183.893
182	3.940	2183.975
183	3.947	2184.129
184	3.958	2184.396
185	3.963	2184.504
186	3.971	2184.704
187	3.986	2185.051
188	3.992	2185.190
189	4.003	2185.451
190	4.008	2185.555
191	4.016	2185.750
192	4.019	2185.829
193	4.025	2185.975
194	4.036	2186.228
195	4.041	2186.330
196	4.049	2186.520
197	4.062	2186.850
198	4.068	2186.982
199	4.079	2187.229
200	4.083	2187.328
201	4.091	2187.513
202	4.104	2187.834
203	4.110	2187.963
204	4.120	2188.203
205	4.125	2188.300
206	4.132	2188.480
207	4.145	2188.793
208	4.151	2188.919
209	4.161	2189.153
210	4.165	2189.247
211	4.172	2189.423
212	4.185	2189.728
213	4.191	2189.850
214	4.200	2190.078
215	4.204	2190.169
216	4.211	2190.340
217	4.215	2190.409
218	4.220	2190.537
219	4.229	2190.759
220	4.233	2190.848
221	4.240	2191.014
222	4.243	2191.081
223	4.249	2191.206
224	4.258	2191.422
225	4.262	2191.509
226	4.268	2191.671
227	4.271	2191.736
228	4.276	2191.857
229	4.285	2192.068
230	4.289	2192.153
231	4.296	2192.311
232	4.299	2192.374
233	4.304	2192.492
234	4.312	2192.698
235	4.316	2192.780
236	4.322	2192.934
237	4.325	2192.996
238	4.330	2193.111
239	4.338	2193.311
240	4.342	2193.391
241	4.348	2193.541
242	4.359	2193.800
243	4.364	2193.904
244	4.366	2193.949
245	4.369	2194.033
246	4.376	2194.179
247	4.386	2194.432
248	4.391	2194.533
249	4.399	2194.723
250	4.402	2194.799
251	4.408	2194.941

252	4.411	2194.998
253	4.415	2195.104
254	4.423	2195.289
255	4.426	2195.363
256	4.432	2195.501
257	4.442	2195.740
258	4.447	2195.836
259	4.454	2196.014
260	4.457	2196.086
261	4.463	2196.220
262	4.473	2196.452
263	4.477	2196.545
264	4.485	2196.719
265	4.488	2196.789
266	4.493	2196.919
267	4.496	2196.971
268	4.500	2197.069
269	4.507	2197.239
270	4.510	2197.307
271	4.515	2197.434
272	4.525	2197.654
273	4.529	2197.742
274	4.536	2197.907
275	4.539	2197.973
276	4.544	2198.096
277	4.553	2198.310
278	4.557	2198.396
279	4.564	2198.557
280	4.567	2198.621
281	4.572	2198.741
282	4.574	2198.790
283	4.578	2198.880
284	4.585	2199.036
285	4.587	2199.099
286	4.592	2199.216
287	4.595	2199.263
288	4.598	2199.351
289	4.605	2199.503
290	4.616	2199.766
291	4.621	2199.871
292	4.629	2200.069
293	4.633	2200.148
294	4.639	2200.296
295	4.642	2200.355
296	4.647	2200.457
297	4.654	2200.626
298	4.662	2200.812
299	4.675	2201.128
300	4.681	2201.254
301	4.691	2201.490

TABLE A.4.4 - 315 DEG. APPROACH INELASTIC EVENT TABLE

P Inelastic Event Detailed Report
39:59 1994

Sat Aug 20

oject: ss224 Model: model1 Version: 1

ber Name	Load Step	Time	Element Type	Event Description
a2-60	21	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
1-97	49	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
1-97	54	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
1-97	291	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
1-97	292	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)

A.6 Fixed Base Ultimate Strength Analysis

An ultimate strength CAP analysis was performed on the benchmark platform with the pile elements below the mudline removed and the leg joints at the mudline fixed. The diagonal direction (315 deg.) was analyzed because this was the critical direction for the piled base model and because this direction produced the lowest lateral load required for 1st element yield in the jacket (S1).

Results of the analysis are shown in Figure A.6.1 which shows the fixed base results together with the piled base results. The fixed base results are tabulated in Table A.6.1. Table A.6.2 lists the component failures and inelastic events for this analysis. The ultimate capacity for the fixed base model is expected to be slightly higher than the values shown on Figure A.6.1 because the capacity curve was continuing to increase at a slow rate when the analysis was terminated (due to time constraints). Full failure mechanisms (plastic hinges) had not yet formed in analysis when it was terminated. The critical failure components were found to be the leg members which were modeled as beam columns. The deflected shape of the fixed base model is shown in Figure A.6.2.

FIGURE A.6.1 — 315 DEGREE DIRECTION FIXED & PILED BASE

Load at 1st component I.R.=1.0 (S1–fix) = 1100 kips

100 year, 20th ed. ref. load (Sref) = 1310 kips

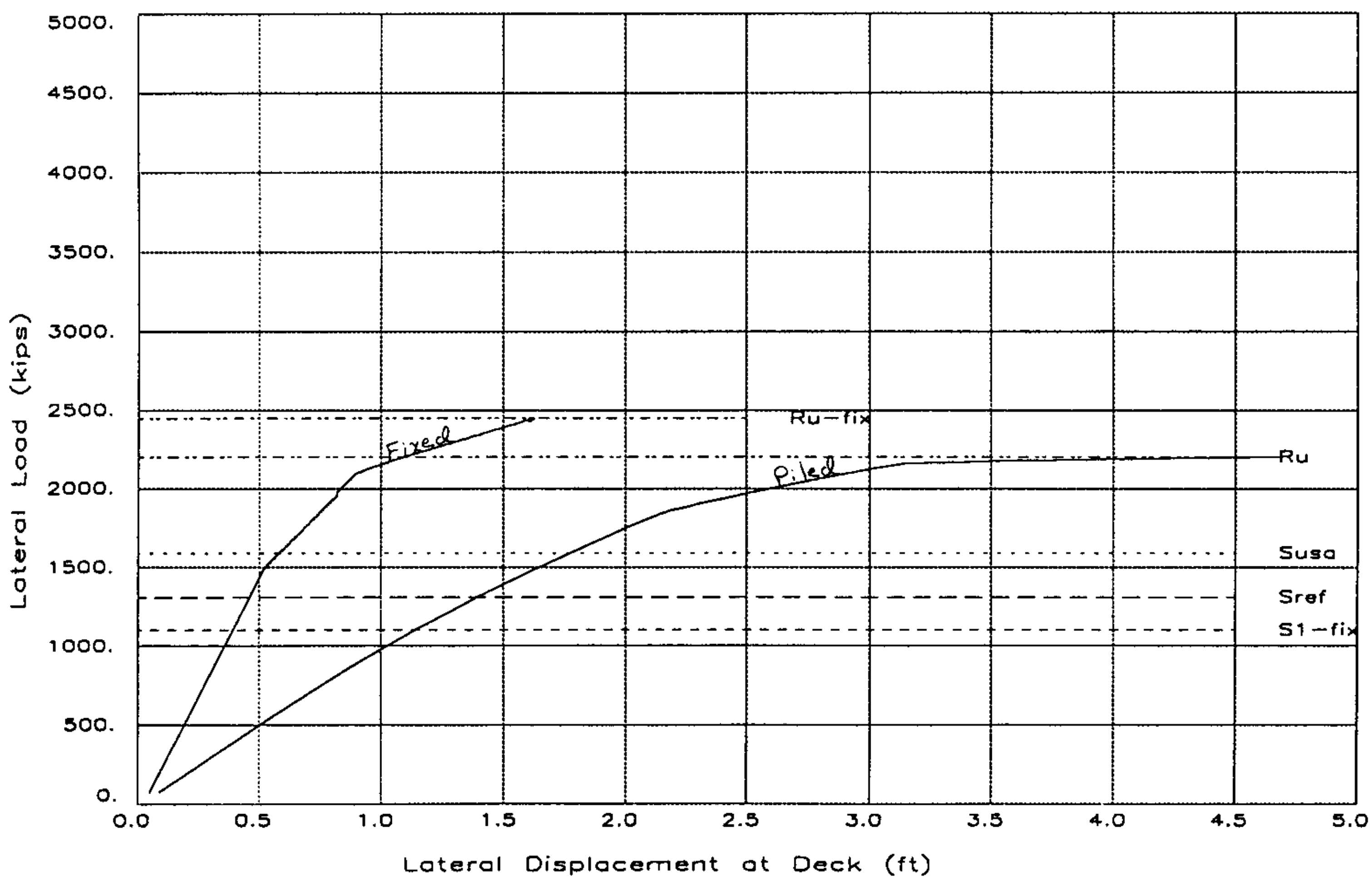
Ultimate Strength Analysis Load (Susa) = 1590 kips

Ultimate Capacity (Ru) = 2200 kips

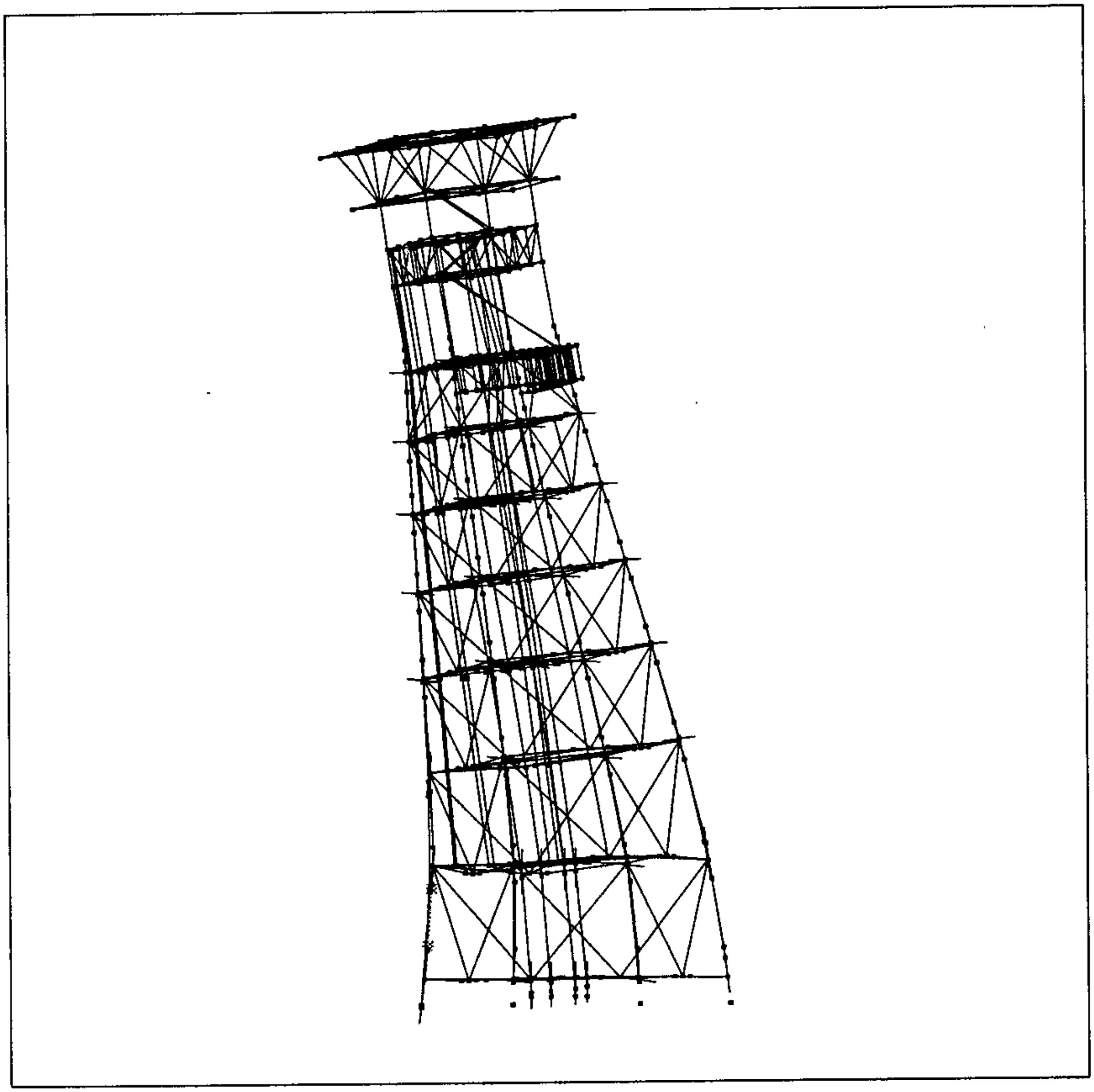
Ultimate Capacity Fixed (Ru–fix) = 2446 kips

Reserve Strength Ratio = 1.9 (fixed)

Platform Failure Mode = Foundation



Project: fixbase Model: model1 Version: 0   Tue Aug 23 13:53:13 1994 *



CAP z ↑ → x   Figure A.6.2 Fixed Base deflected shape

Inelastic Events Legend

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

*** TABLE A.6.1 FIXED BASE 315 DEG **

CAP Results Table — Tue Aug 23 08:13:36 1994

Project: fixbase Model: model1 Version: 0

Cut Plane Force Fx - Kip

fixed base 315 deg.

Load Step	X Deck Disp Ft	Force Kips
0	000.000e-3	000.000e-3
1	53.977e-3	72.459
2	0.255	683.828
3	0.455	1294.972
4	0.505	1447.589
5	0.511	1464.078
6	0.520	1492.135
7	0.523	1495.167
8	0.526	1497.250
9	0.529	1502.781
10	0.532	1510.500
11	0.535	1516.225
12	0.538	1521.603
13	0.542	1526.483
14	0.545	1531.377
15	0.548	1536.245
16	0.551	1541.143
17	0.554	1546.010
18	0.557	1550.907
19	0.561	1555.774
20	0.564	1560.672
21	0.567	1565.538
22	0.570	1570.436
23	0.573	1575.302
24	0.577	1580.199
25	0.580	1585.065
26	0.583	1589.962
27	0.586	1594.828
28	0.589	1599.725
29	0.592	1604.591
30	0.596	1609.488
31	0.599	1614.354
32	0.602	1619.250
33	0.605	1624.117
34	0.608	1629.013
35	0.611	1633.879
36	0.615	1638.776
37	0.618	1643.642
38	0.621	1648.538
39	0.624	1653.405
40	0.627	1658.301
41	0.630	1663.167
42	0.634	1668.063
43	0.637	1672.930
44	0.640	1677.826
45	0.643	1682.692
46	0.646	1687.588
47	0.650	1692.455
48	0.653	1697.351
49	0.656	1702.218
50	0.659	1707.114
51	0.662	1711.980
52	0.665	1716.876
53	0.669	1721.743
54	0.672	1726.639
55	0.675	1731.506
56	0.678	1736.401
57	0.681	1741.268
58	0.684	1746.164
59	0.688	1751.031
60	0.691	1755.926
61	0.694	1760.793
62	0.697	1765.689
63	0.700	1770.556
64	0.704	1775.452
65	0.707	1780.319
66	0.710	1785.215
67	0.713	1790.082
68	0.716	1794.977
69	0.719	1799.844
70	0.723	1804.740
71	0.726	1809.607
72	0.729	1814.503
73	0.732	1819.370
74	0.735	1824.266
75	0.738	1829.132
76	0.742	1834.028
77	0.745	1838.895
78	0.748	1843.791
79	0.751	1848.658
80	0.754	1853.553
81	0.757	1858.420
82	0.761	1863.316
83	0.764	1868.183
84	0.767	1873.079
85	0.770	1877.946
86	0.773	1882.842
87	0.777	1887.709
88	0.780	1892.604
89	0.783	1897.472
90	0.786	1902.367
91	0.789	1907.234
92	0.792	1912.130
93	0.796	1916.997
94	0.799	1921.893
95	0.802	1926.760
96	0.805	1931.655
97	0.808	1936.522
98	0.811	1941.418
99	0.815	1946.285
100	0.818	1951.181
101	0.821	1977.533
102	0.824	1982.480
103	0.827	1987.396
104	0.831	1992.343
105	0.834	1997.259
106	0.837	2002.205
107	0.840	2007.122
108	0.843	2012.068
109	0.846	2016.985
110	0.850	2021.931
111	0.853	2026.848
112	0.856	2031.794
113	0.859	2036.711
114	0.862	2041.657
115	0.865	2046.574
116	0.869	2051.520
117	0.872	2056.437
118	0.875	2061.337
119	0.878	2066.219
120	0.881	2071.061
121	0.884	2075.832
122	0.888	2080.612
123	0.891	2085.360
124	0.894	2090.124
125	0.897	2094.762
126	0.900	2095.413
127	0.904	2097.937
128	0.907	2101.362
129	0.910	2102.204
130	0.913	2101.992
131	0.916	2105.120
132	0.920	2108.505
133	0.923	2110.455
134	0.926	2112.034
135	0.929	2114.002
136	0.933	2115.515
137	0.936	2117.572
138	0.939	2118.982
139	0.942	2121.164
140	0.945	2122.425
141	0.949	2124.780
142	0.952	2125.831
143	0.955	2128.344
144	0.958	2129.049
145	0.961	2130.520
146	0.963	2131.953
147	0.966	2132.967
148	0.969	2135.867
149	0.973	2136.743
150	0.976	2137.884
151	0.979	2139.462
152	0.982	2140.833
153	0.985	2143.250
154	0.989	2145.154
155	0.992	2147.310
156	0.995	2150.270
157	0.998	2151.296
158	1.001	2153.501
159	1.005	2154.698
160	1.008	2156.727
161	1.009	2157.285
162	1.013	2156.742
163	1.016	2157.725
164	1.019	2161.574
165	1.022	2163.997
166	1.025	2164.262
167	1.029	2167.001
168	1.032	2169.145
169	1.035	2170.623
170	1.038	2173.210
171	1.041	2175.502
172	1.045	2176.983
173	1.048	2179.054
174	1.051	2180.136
175	1.054	2182.147
176	1.058	2183.191
177	1.061	2185.290
178	1.064	2185.655
179	1.067	2187.753
180	1.071	2186.094
181	1.074	2184.015
182	1.077	2189.686
183	1.080	2192.694
184	1.083	2195.052
185	1.087	2196.173
186	1.090	2197.961
187	1.093	2199.748
188	1.096	2202.001
189	1.100	2203.333
190	1.103	2205.147
191	1.106	2204.754
192	1.109	2205.788
193	1.113	2207.709
194	1.116	2210.336
195	1.119	2211.994
196	1.122	2214.173
197	1.126	2215.303
198	1.129	2217.390
199	1.132	2217.854
200	1.135	2219.310
201	1.138	2220.844
202	1.142	2221.258
203	1.145	2223.788
204	1.148	2223.459
205	1.151	2225.896
206	1.155	2227.960
207	1.158	2230.570
208	1.161	2231.052
209	1.164	2232.908
210	1.168	2234.167
211	1.171	2236.486
212	1.174	2237.219
213	1.177	2238.428
214	1.181	2240.069
215	1.184	2242.289
216	1.187	2243.249
217	1.190	2245.077
218	1.194	2245.800
219	1.197	2247.164
220	1.200	2249.573
221	1.203	2251.931
222	1.206	2252.615
223	1.210	2255.442
224	1.213	2254.924
225	1.216	2254.797
226	1.219	2259.053
227	1.223	2259.881
228	1.226	2262.834
229	1.229	2263.333
230	1.232	2265.696
231	1.236	2266.084
232	1.239	2268.230
233	1.242	2269.269
234	1.245	2270.500
235	1.249	2272.085
236	1.252	2275.083
237	1.255	2274.830
238	1.258	2275.945
239	1.262	2277.767
240	1.265	2279.783
241	1.268	2281.013
242	1.271	2283.394
243	1.275	2283.270
244	1.278	2283.107
245	1.281	2287.196
246	1.284	2286.479
247	1.287	2288.096
248	1.291	2290.704
249	1.294	2293.153
250	1.297	2295.286

251	1.300	2297.555
252	1.304	2298.193
253	1.307	2300.934
254	1.310	2300.117
255	1.313	2300.713
256	1.317	2304.238
257	1.320	2306.099
258	1.323	2307.275
259	1.326	2309.228
260	1.330	2310.234
261	1.333	2312.940
262	1.336	2311.944
263	1.339	2311.825
264	1.343	2315.343
265	1.346	2313.564
266	1.349	2316.417
267	1.352	2320.216
268	1.356	2321.673
269	1.359	2323.715
270	1.362	2325.285
271	1.365	2327.330
272	1.368	2328.604
273	1.372	2330.582
274	1.375	2331.375
275	1.378	2333.923
276	1.381	2333.548
277	1.385	2335.453
278	1.388	2337.195
279	1.391	2339.415
280	1.394	2339.874
281	1.398	2341.741
282	1.401	2342.227
283	1.404	2342.610
284	1.407	2345.875
285	1.411	2347.910
286	1.414	2349.413
287	1.417	2351.817
288	1.420	2352.196
289	1.423	2354.882
290	1.427	2354.011
291	1.430	2353.650
292	1.433	2357.584
293	1.436	2359.468
294	1.440	2361.610
295	1.443	2363.268
296	1.446	2364.807
297	1.449	2366.463
298	1.453	2367.783
299	1.456	2369.589
300	1.459	2370.614
301	1.462	2372.859
302	1.466	2373.211
303	1.469	2375.856
304	1.472	2375.546
305	1.475	2376.876
306	1.479	2379.225
307	1.482	2380.618
308	1.485	2380.966
309	1.488	2382.669
310	1.492	2384.401
311	1.495	2387.220
312	1.498	2387.768
313	1.501	2388.716
314	1.505	2390.987
315	1.508	2390.917
316	1.511	2394.051
317	1.514	2395.921
318	1.517	2398.196
319	1.521	2398.913
320	1.524	2401.607
321	1.527	2400.914
322	1.530	2401.855
323	1.534	2405.234
324	1.537	2406.688
325	1.540	2407.581
326	1.543	2408.189
327	1.547	2411.326
328	1.550	2412.310
329	1.553	2414.340
330	1.556	2415.029
331	1.560	2417.279
332	1.563	2417.624
333	1.566	2418.015
334	1.569	2420.928
335	1.572	2423.199
336	1.576	2424.992
337	1.579	2426.396
338	1.582	2427.855
339	1.585	2428.859
340	1.589	2431.104
341	1.592	2431.848
342	1.595	2433.479
343	1.598	2433.398
344	1.602	2436.158
345	1.605	2430.768
346	1.608	2425.942
347	1.611	2432.216
348	1.614	2433.075
349	1.618	2435.041
350	1.621	2441.189
351	1.624	2445.275
352	1.627	2445.985

*** TABLE A.6.2 *** FIXED BASE

CAP Inelastic Event Detailed Report Tue Aug 23 08:13:36 1994

Project: fixbase Model: modell Version: 0

Member Name	Load Step	Time	Element Type	Event Description	
lgbl-81	6	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-44	118	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-44	120	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-85	151	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-85	154	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-85	155	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-85	163	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-85	164	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	165	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	166	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	167	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-85	168	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	178	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	179	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	180	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	183	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	191	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	193	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-85	196	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgbl-85	197	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	202	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	203	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	208	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	209	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	219	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	220	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	224	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	226	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	234	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	235	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	237	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	238	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	243	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	245	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	254	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	256	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	262	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	264	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-40	270	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	276	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	277	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	280	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	281	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-40	283	000.000e-3	Beam Column	Elastic	(0,0)
lga2-40	284	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-40	290	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	290	000.000e-3	Beam Column	Elastic	(0,0)
lga2-40	291	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	292	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	304	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	305	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgbl-77	306	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	308	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgbl-77	309	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgbl-77	314	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-40	315	000.000e-3	Beam Column	Elastic	(0,0)
lga2-40	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-40	318	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	321	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	323	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	333	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	334	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-40	345	000.000e-3	Beam Column	Elastic	(0,0)
lga2-40	346	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-40	347	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-77	347	000.000e-3	Beam Column	Elastic	(0,0)
lga2-40	348	000.000e-3	Beam Column	Elastic	(0,0)
lgbl-77	348	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-40	349	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgbl-77	349	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-40	350	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-48	352	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgbl-89	352	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)

PARTICIPANT "E"

1.0 SUMMARY

The ultimate strength of the benchmark structure is evaluated using two structural programs, SEADYN-SPLICE and the Extended Design Program, EDP. The SEADYN-SPLICE analysis models a linear structure, linear piles with nonlinear soils. The EDP analysis includes nonlinear structural members (beam-columns and struts), nonlinear joints and fully nonlinear piles, conductors and soils. The two independent approaches yield critical reserve strength ratios between 0.72 and 0.74 of the design East loading of 3230 kips. Both approaches result in compression soil failure which subsequently causes portal pile failures and platform collapse.

Another EDP analysis for the diagonal NE direction results in a marginally higher RSR of 0.76 with a reference base shear of 2780 kips and also culminates with compression pile plunging and tension pile pullout. The SE direction has a base shear of 2360 kips and would be expected to yield an RSR of 0.89 (17% higher than the NE). The other loading directions have base shears of 1710 (N), 1310 (S), 1030 (NW), 600 (W) and 520 (SW) kips and would be expected to have RSRs of 1.40, 1.82, 2.05, 3.48 and 4.06, respectively. The latter RSRs are estimated based on the East and NE analyses.

The linear SEADYN structural model includes all jacket structural elements, major deck framing and the guided piles within the jacket legs. Modelling detail includes joint cans and conductor guide framing. All members are modelled as linear beams. Risers, conductors and boatlandings are included for wave load generation only. The pile response is solved using a substructuring technique to condense the jacket stiffness and loads to equivalent matrices for the nonlinear soil SPLICE analysis. The SPLICE displacement results at the pile tops (below jacket) are then backsubstituted to derive the linear jacket response for subsequent API code checks.

The nonlinear EDP model is a fully nonlinear coupled jacket/pile/soil model. The conductors are also included as nonlinear beam-columns that are guided at the plan frames and laterally resisted by nonlinear P-Y soil elements. The jacket model maintains the complete detail of the linear model with all members below the decks modelled nonlinearly. All K-joints in the vertical frames are modelled for convenience as rigid-plastic action-deformation components. Other less critical joints are not modelled. The K-brace diagonals are modelled as axial strut elements. All other jacket elements are modelled as nonlinear beam-columns. Nonlinear piles and surrounding soils are included. In addition, the jacket leg extensions below the mudline plan are modelled with gap elements between the jacket and the piles. The surrounding soils for these 12 ft extensions bear on the jacket legs, not the internal piles.

In participating in this benchmark effort, it has been our understanding that the goal was to implement the proposed API RP2A 20th edition Section 17. Specifically, an ultimate strength RSR less than 1.0 does not satisfy the requalification criteria. Since the soil failure occurs below this requalification threshold, the only value in performing a nonlinear structure analysis would be to predict a lower RSR value. Although minor yielding in joints and the conductor framing occur prior to the soil failure, the practical value of such an observation is that the structure still fails requalification.

The PMB optional task of "fixed-base case (assuming no piles and the jacket fixed at the seabed)" seems unrealistic in that the structure was originally designed as a jacket not a tower. Perhaps a more reasonable suggestion would have been to increase the soil capacities to make a more interesting structural performance and a better benchmark for the industry.

A.1 ENVIRONMENTAL CRITERIA

The metocean criteria for the ultimate strength analysis are based on guidance of the "API RP2A-WSD, 20th Edition" (API RP2A 20th Edition) and "API RP2A Draft Section 17, Assessment of Existing Platforms" (Draft Guidelines) for Gulf of Mexico region.

The water depth criteria are:

MSL Water Depth	157 feet
Storm + Astronomical Tide	3 feet
Design Water Depth	160 feet

The design wave height, period and associated currents are summarized in Table A.1.1-1 for eight wave approach directions. A brief description of the major metocean criteria and their source are described in the following sections.

A.1.1 Wave Height and Period

The base wave height of 68 feet is determined from Figure 17.6.2-2a of the Draft Guidelines for a full population hurricane. The directional wave heights shown in Table A.1.1-1 are based on the directional rosette shown in Figure 17.6.2-4 of the Draft Guidelines.

The wave period of 13.5 seconds is obtained from Table 17.6.2-1 of the Draft Guideline. The apparent wave period is calculated for each wave direction according to procedures outlined in Section 2.3.1b of API RP2A to account for the Doppler shift due to current. The in-line current (V_I) and wave period (T) are used in Figure 2.3.1-2 of API RP2A 20th Edition to determine the apparent period, T_{app}.

Section 2.3.1b-3 of API RP2A 20th Edition allows for a wave kinematics factor, which reduces horizontal wave velocities and accelerations, to account for wave directional spreading. For analysis simplicity, a wave kinematic factor of 0.88 is applied to all components of the wave velocities and accelerations. As the primary load component is horizontal, application of the factor to the vertical velocities and accelerations is judged to be negligible in terms of global structure loads.

A.1.2 Current

A constant current profile of 2.3 knots (3.89 ft/sec) is taken from Table 17.6.2-1 of the Draft Guideline and Figure 2.3.4-6 of API RP2A 20th Edition. Procedures outlined, for the intermediate water depth zone, in Section 2.3.4c of the API RP2A 20th Edition are used to calculate current velocities and directions for each of the wave approach directions.

The intermediate water depth zone requires interpolation of the shallow and deep water zone current velocities and direction.

Using API RP2A 20th Edition, the current direction for the shallow zone is constant. The shallow water current direction is 235 degrees measured clockwise from Platform North.

For the deep water zone the current vector is fully in-line with the wave direction, and is adjusted using the directional wave height factors shown in Figure 17.6.2-4 of the Draft Guidelines.

The current magnitude and direction listed in Table A.1.1-1 are for an intermediate water depth zone and result from interpolation of component values between the shallow and deep water zones.

A.1.3 Deck Wave

Deck wave and current loads are determined using the procedure described in the Section C17.6.2 of the Draft Guildelines. The wave height and crest velocity are determined for each of the wave directions of Table A.1.1-1 using Stokes 5th order wave theory. The deck wave force is then calculated in accordance with the Draft Guidelines for the wetted "silhouette" area (A), as follows:

$$F_{dk} = (1/2)\ \rho\ C_d\ (\alpha_{wkf}{*}V + \alpha_{cbf}{*}U)^2\ A$$

where U is the current speed in-line with the wave, V is the wave particle speed, α_{wkf} is the wave kinematics factor (0.88 for hurricanes), α_{cbf} is the current blockage factor (taken as 0.80 for broadside current and 0.85 for diagonal current as per Section 2.3.1b-4 of API RP2A 20th Edition), C_d is the drag coefficient for the jacket and ρ is the density of sea water.

The drag coefficients used for the deck wave force calculation are given in Table C17.7.2-1 of the Draft Guidelines.

A.1.4 Wind Speed

The wind speed of 85 knots (143.8 ft/sec) at a reference elevation of 10.0 meters, extracted from Table 17.6.2-1 of the Draft Guidelines, is used to determine the wind loads on the deck area.

A.1.5 Hydrodynamic Coefficients

The hydrodynamic coefficients for unshielded cylindrical members are used to determine wave forces. The drag and inertia coefficients for members below the waterline are 1.05 and 1.2, respectively, as specified in Section 2.3.1b-7 of API RP2A 20th Edition. The drag coefficients were reduced to 0.70 for members above the waterline.

A.1.6 Marine Growth

A full marine growth profile of 1.5 inches is used as specified in Section 2.3.4d of API RP2A 20th Edition.

A.1.7 Environmental Loads

The wave loads are based on conventional Morison's equation accounting for the drag and inertial forces using the vector sum of the wave current kinematics.

The loads shown in Table A.1.7-1 show the combined wave, current and wind loads, and the deck wave loads for the directions considered.

A.2 3-D MODEL GENERATION

The analytical models are presented in this section. The linear structure-nonlinear soil model is presented in Section A.2.1 and the fully nonlinear model is presented in Section A.2.2.

A.2.1 Linear Structure-Nonlinear Soil

A.2.1.1 Structural Model

The 3D space-frame computer model consists of all the major structural components including the jacket legs, piles inside the legs, horizontal bracing with main conductor framing members, K-bracing in the frames of the jacket, and all major deck components. The appurtenances including conductors, risers, boatlandings, barge bumpers, and stairways are not included in the linear structural model, but are included in hydrodynamic modeling of the loads. The primary jacket structure joint can thicknesses and stub thicknesses are included, as are member end eccentricities, including those less than 25% of the chord diameter. The lack of mudmat information prevented their inclusion in the model. As the foundation model (see Section A.2.1.2) accounts for the structure below the mudline, the jacket model ends at the mudline.

The topsides model includes the primary steel members framing in sufficient detail to correctly represent the stiffness and load paths. Nodal eccentricities are not modelled for the topsides.

The pile sections above the mudline are rigidly attached to the legs at the crown shims and guided at each plan level elevation transferring only lateral forces.

The effective length factors for the K-Brace diagonals of 0.7 versus the recommended API RP2A 20th Edition of 0.8 is used to better represent the actual member stiffness and reduce the conservatism in using a higher factor.

The member capacities are evaluated using the API RP2A 20th Edition code conformance equations using an allowable stress factor of 1.7 for the given yield stress of 42 ksi.

A.2.1.2 Foundation Model

The nonlinear soil model is generated using soil data provided by PMB. The lateral load-displacement (P-Y) data are generated using the methods of Reese, Cox and Koop (1974) for sand, Matlock (1970) soft clay and Reese (1975) for hard clay. The axial load-displacement (T-Z) data and tip load-displacement (Q-Z) data are generated using the recommendations in Section 6.7.2 and 6.7.3, respectively, of API RP2A 20th Edition.

The foundation is modelled using a full non-linear/structure interaction treating the jacket as linear with nonlinear foundation. Using this approach, the jacket model (excluding wave loading members) is reduced to an equivalent boundary representation (stiffness matrix and condensed load vectors) at the bottom of each leg. The nonlinear pile/soil interaction program SPLICE is then used to compute the pile group displacement at the mudline level. The total solution in the structure and pile is then solved by backsubstituting the boundary node displacements into the SEADYN model.

A.2.2 Nonlinear EDP Model

The nonlinear EDP model is a fully nonlinear coupled structure and foundation model as shown in Figure A.2.2-1. The jacket is modelled as a fully nonlinear system below the decks. The complete detail used for the linear representation has been retained, with the addition of the conductors horizontally guided by conductor framing. The decks are modelled as presented previously for the linear model. The foundation includes the axial and lateral behavior of the four main piles and the lateral behavior of the conductors.

The EDP nonlinear modelling process performed for this study is a result of progressive modelling refinements. The models created and studied include:

- o Linear substructure - linear matrix foundation
- o Linear substructure - nonlinear foundation
- o Nonlinear substructure - nonlinear foundation
- o Nonlinear substructure with leg extensions - nonlinear foundation
- o Nonlinear substructure, leg ext. and joints - nonlinear foundation

The incremental refinement process is used to the analysts' advantage to clearly and efficiently interpret nonlinear response and is not a requirement for EDP applications. The subsequent discussion presents the modelling assumptions and techniques used in the final model which corresponds with the results presented.

A.2.2.1 Nonlinear Jacket Model

All structural elements in the jacket model are modelled as nonlinear beam-column or strut members. The K-joints in the vertical frames are modelled for convenience as rigid-plastic action-deformation components (flexibility could have been included). In addition, the annulus between the leg extensions (below the mudline plan) and the piles are modelled as nonlinear gap elements. Lateral interactions between the piles and the legs are accommodated with shim constraints that permit axial slip without lateral translation. Lack of information about the mudmats prevent their inclusion in the model. P-delta geometric effects are included in the jacket members.

The main vertical frame diagonals are modelled as buckling strut elements. The member buckling values account for local distributed lateral loads from dead, buoyancy and storm loads. The slenderness values assigned are 0.7. For the analyses performed, the K-joint capacities are smaller than the buckling capacities which precludes these members from buckling. The K-joint modelling is described subsequently.

All other structural members are modelled as nonlinear beam-column elements which yield with concentrated plastic hinges at each end based on the interaction of axial, bending and torsional member forces. The axial tension yield is defined to be the full section yield force based on 42 ksi. The compressive force is the buckling force of the member associated with the longest effective length. The post axial yield stiffness is assigned to be two percent of the original elastic value. Initial and full bending yields are defined to be the extreme fiber and full plastic section moments, respectively. The first and full yield stiffness are assigned as 30% and 2% of the initial stiffness, respectively.

The 12.0 ft jacket leg extensions below the mudline plan are modelled as nonlinear beam-column elements with similar response characteristics to other jacket members. Gap elements connect the three extension segments on each leg to the enclosed pile. Initially the leg is assumed to be separated from the pile by the 1/2 inch gap annulus. The surrounding tributary soils are attached to these leg extensions both for lateral P-Y and axial T-Z soil resistances. The internal pile does not notice the influence of the soils until the gap closes between the pile and the leg.

Joint Modelling and Capacities:

The main frame vertical K-joints are modelled as nonlinear rigid-plastic members which yield at the joint capacities, potentially limiting the loads developed in the vertical diagonal braces. From a global performance perspective, the yielded plastic joint modelling also limits the shear transfer through the K-brace subassemblage. The joint capacities, which are based on the lower bound API RP2A section 4 nominal brace capacities, are used to exaggerate the effects of joint yielding for these analyses. In an actual requalification effort, values based on mean capacities would be used.

Initially, EDP linear analyses are used to evaluate all joints in the jacket for each directional loading condition. Within EDP, the automatic load based joint classification is performed according API RP2A 20th edition. These analyses indicate that the K-joints in the vertical main frames would attain their lower bound yield estimates at 50% of the more critical East storm loading. Other joints and other loading directions have much smaller utilizations.

Within EDP, joints are defined with both flexibility and capacities. These gapped K-joints have a branch to chord diameter ratio (beta) of 1.0 which tends to minimize chord shell bending flexibility. The axial load balance between the tension and compression braces also acts to stabilize the chord member and inhibits ovalization. These effects justify the modelling simplification of the rigid joint. The ultimate joint capacities are computed using the lower bound joint capacity specified in RP2A Section 4 equations 4.3.1-4a and 4.3.1-4b, and a stress increase factor of 1.70.

A.2.2.2 Nonlinear Foundation Model

The EDP foundation models for the piles and conductors include the nonlinear steel pipe sections and soil reaction springs for soil lateral and frictional resistance. The axial discretization of each pile-soil model is chosen to accurately represent soil layering and pile section changes. Approximately 18 segments are used for both the conductors and the piles below mudline. The nonlinear pile models parallel the legs and are laterally guided at plan frames from the bottom of the leg extensions to the rigid crown shim connection. The conductors are laterally supported by the conductor framing at about half the plan frames in accordance with the structure design.

The piles and conductors are modelled using nonlinear beam-column segments which form hinges based on a coupled interaction surface which relates axial and bending forces. The axial force-deformation characteristics are elastic up to the full section axial capacity with two percent post-yield strain hardening. Initial bending yield is defined to be first yield of the extreme fiber. Full yield is defined as the full plastic moment of the section. A reduced bending stiffness of one third the initial stiffness is used between first yield and full plastic moments. Subsequent bending yield beyond full plastic is defined as two percent of the initial. In addition, P-delta geometric effects are included.

The soils are defined as multilinear reaction springs which represent the frictional and lateral bearing resistance of the tributary soils at each pile node. The soil properties have been defined to be consistent with the linear modelling described, previously. The axial friction between the soil and the conductors has been ignored. Tip bearing is defined at the bottom of each of the main piles.

A.3 SOFTWARE DESCRIPTION

A.3.1 SEADYN

The SEADYN Ocean Structures Design and Analysis Language, used for the linear structural analysis, is an ICES subsystem with specific enhancements for marine and offshore engineering applications. Developed by Kvaerner Earl and Wright, formerly Earl and Wright Consulting Engineers, SEADYN is designed to perform structural analyses and code conformance checks.

A.3.2 SPLICE

The SPLICE program is a soil/pile interaction program developed by Norwegian Geotechnical Institute and Aker Engineering A/S. The program solves the combined system of a linear elastic superstructure and its nonlinear piled foundation system for displacements of the pile/structure interface points. A complete set of pile solutions (displacement, forces, moments and stresses with depth) is found by back-substitution of the interface solution through the piles. The superstructure may be solved by a similar back-substitution, using a program other than SPLICE.

A.3.3 PILEPYG

The PILEPYG program, developed by Kvaerner Earl and Wright, is a lateral force-displacement curve generation program for a given pile/soil system at regular depths below the mudline. Conventional methods described by Reese, Cox and Koop (1974) for sand, Matlock (1970) for soft clay, and Reese (1975) for hard clay are used to generate the P-Y data for use in other software packages.

A.3.4 EDP

Extended Design Program, EDP, has been developed by Digital Structures Inc. since 1980 for the nonlinear analysis of three dimensional nonlinear structures and foundation systems. EDP is designed for efficient static and dynamic analysis of fixed and compliant offshore platforms, bridges, piers and buildings. Versions exist for computers ranging from mainframes to PCs.

EDP uses the direct stiffness method to assemble the stiffness and mass matrices from the properties of individual finite elements. A tangent stiffness formulation with incremental loads is used to determine nonlinear response. Constant stiffness and Newton Raphson iteration schemes are among those used for both static and dynamic analyses to maintain internal equilibrium with the externally applied loadings and to minimize numerical instability. Nonlinear dynamic response is determined by direct integration of the equations of motion. Linear dynamic analyses may be time or frequency domain solutions. Nonlinear static analysis loads and displacements can be manually incremented or automatically applied.

Local failures of structure and soil elements can be determined with advancing time. Progressive changes in geometry are also reflected in the analysis. Failure of elements may be defined by force limits, deformation limits, cumulative deformation or by hysteretic energy dissipation. Time varying nodal forces, displacement functions, hydrodynamic loads or ground acceleration time histories (including phased support motions) can be used as structure loadings. Preload, loading and unloading are available. Element environmental loads (dead, buoyancy, wave etc.), local loads and mass (material, marine growth, contained water and hydrodynamic) are automatically generated and may be combined with user-defined nodal quantities. Regular or random sea wave load histories can be generated.

The available modelling and numerical techniques permit accurate and reliable representation of nonlinear behavior due to; member and joint element material yield, buckling, hysteretic behavior, large displacements, gapped joints including impact, friction, constrained deformations, energy dissipation, relative motion kinematic effects, strength degradation and radiation damping effects in soils. Structural elements can be modelled using linear beam, strut, two or three hinge beam-columns, plate, shell, solid, gap, friction, nonlinear spring and matrix representations. Hybrid joint-beam elements elegantly model the nonlinear response of tubular joints in offshore platforms for strut buckling and inelastic beam-column elements. Nonlinear spring algorithms can replicate any physical force-deformation behavior, including elastic or hysteretic responses, allowing for impact, uplift and friction.

A complete range of soils and foundation responses can be modelled using either multilinear elastic or hysteretic elements, as applicable, for study of cyclic effects, including degradation and gapping.

Integrated postprocessing and graphics capabilities assist in the interpretation and documentation of results. Tabular or graphical forms of output are available for all input parameters and responses obtained during analyses. These include displacements, forces, stresses and code check utilizations resulting from static dynamic analyses. Dynamic analyses may be presented as animated displaced shapes or composite plots of displaced shapes.

A.4. ULTIMATE STRENGTH ANALYSIS

This section presents two ultimate strength analyses, one with a linear structure and nonlinear soil (Refer Section A.4.1) and the other with both nonlinear structure and soil (Refer Section A.4.2). A reference analysis using both a linear structure and foundation is used to calibrate the nonlinear model and discussed briefly in the latter section.

A.4.1 Linear Structure-Nonlinear Soil Ultimate Strength Analysis

The most severe direction of loading, loading from Platform East, was selected for the analysis. The linear analysis results are presented in Table A.4.1-1. The calculated reserve strength ratio is 0.72. This value is obtained by dividing the ultimate capacity lateral load of the jacket foundation, 2330 kips, by the reference level lateral load of 3230 kips. Figure A.4.1 shows the plot of the lateral load versus displacement.

Local failure occurs in the K-brace joints in Rows 1 and 2. Figures A.4.2, A.4.3, and A.4.4 show the locations and sequence of failure, along with utilizations at the 1890 kips lateral load level. However, this mode of failure does not result in jacket instability. Collapse of the jacket occurs when the piles fail at a load level of 2330 kips.

A.4.2 Nonlinear Ultimate Strength Analysis

The final EDP nonlinear model described previously is a result of progressively refining the analytical model. The EDP linear structure with nonlinear foundation essentially reproduced the results presented in the previous section. Once achieving this result that the soil failure RSR is below 0.75 for the EAST storm, it becomes clear that this structure will not pass the reassessment process defined by the proposed API RP2A Section 17 without at least foundation remedial work. Any further nonlinear model refinement efforts could at best only further reduce the already unsatisfactory RSR value. Additional efforts (modelling nonlinear substructure, leg extensions and nonlinear joints) have been made in the spirit of the joint industry project to further benchmark data. It is rather disappointing that the additional refinements result in the conclusion that the RSR and failure mode are virtually unchanged. This section presents the final East storm analysis results for the fully nonlinear model.

The base shear versus deck displacement for the EAST storm is shown in Figure A.4.2-1. Results from the linear structure with matrix and nonlinear foundations (from previous section) are superimposed on the fully nonlinear solution. The linear matrix solution represents merely an expedient step to obtain member forces, API RP2A member checks and joint checks and as a model verification step. Further discussion of this analysis will not be presented. The second curve (indicated by the circles) for the linear jacket with the nonlinear foundation does not include the leg extensions nor the conductors. The importance of the conductor modelling (3 @ 30 inch and 1 @ 45 inch) can be readily seen in comparison with the fully nonlinear results (triangle). The platform overturning or axial pile soil failure mode and RSR of 0.74 are virtually identical to the results with the linear jacket.

The EDP structure/foundation model deformed shape is shown in Figure A.4.2-2. The exaggerated deck and mudline deflections are 4.0 ft and 1.5 ft, respectively. Minor torsional deformations can be seen in the vertical K-braces of the parallel load resisting frames. Most of the deformation is apparent in the piles below the leg extensions with relatively little flexural deformation within the jacket. The pile segmentation is also readily apparent in this plot.

Axial force versus displacement plots for Row 2 tension and compression piles are shown in Figure A.4.2-3. The initial dead load axial forces are seen to be approximately 600 kips in compression (see superimposed circle and square) with a vertical displacement of 1/2 inch. As the storm load is applied, the tension pile progressively begins to pull out of the soil while the compression pile continues to penetrate. At a compressive deformation of 5.5 inches the compression pile becomes significantly softer until it achieves its ultimate capacity of 3400 kips at a deformation of about 10.0 inches. At this final state, the tension piles still have limited reserve capacity.

The inelastic structural events are tabulated in Table A.4.2-1 and are shown in Figures A.4.2-4 through A.4.2-6. The first structural yield event is an initial yield hinge in the mudline plan frame member 1922 at 0.40 RSR level. Further yield events continue to occur within the bottom plan frame as the load is increased and are attributable to the shear resistance from the conductors. These events are not significant to the structure performance or ultimate RSR attained.

Simultaneous with the first hinge formation in the bottom plan frame at 0.40 load level, the A2 leg extension contacts its internal compression pile. The remaining leg extensions make contact below the 0.50 RSR level. Analyses without the leg extensions result in larger pile bending forces at the bottom plan level which causes limited yielding in the piles. Once the extensions are included, the bending forces reduce and yield is not achieved.

The first K-joint yield occurs at 55 percent of the full storm shear in the tension and compression braces of Row 2 at Level 3, as shown in Figure A.4.2-6. In general, Row 2 has higher loads than Row 1 due to its proximity to the conductors. Subsequent yielding of all the K-joints between Levels 2 through 6 occur in both rows as the storm load in increased. Joint ductilities computed as the ratio of the plastic deformation ductility divided by gamma (D/2t) at the 0.74 RSR level are 7.3 for the most heavily deformed joint at Level 3. Overall these joint inelastic events only marginally change the global force deformation characteristics of the structure and do not effect the RSR attained. Forces on the other joints in the structure at the ultimate strength of the foundation indicate no overstressing.

As the K-joints become plastic, the legs and enclosed piles take any additional shear loading with portal action. The remaining K-braces also contribute to shear resistance. In the later stages of the East pushover, the central K-braces have yielded and only the top and bottom braces remain intact. At the 0.72 RSR level, the compression K-brace at Level 1 begins to buckle. In addition, the legs between Levels 6 and 7 form initial yield hinges.

Loading From Platform	Wave Height (feet)	Wave Period (sec)	Apparent Period (sec)	Current (ft/sec)	Current Angle CW From Platform North (degrees)
North	51.0	13.5	14.03	3.79	233.31
Northeast	61.2	13.5	14.35	3.87	234.58
East	68.0	13.5	14.20	3.86	236.55
Southeast	64.6	13.5	13.68	3.74	237.60
South	57.8	13.5	13.01	3.62	237.00
Southwest	47.6	13.5	12.65	3.58	235.35
West	47.6	13.5	12.85	3.61	233.84
Northwest	47.6	13.5	13.37	3.69	233.06

Table A.1.1-1
Seastate Parameters

Loading From Platform	Wave Height (feet)	Wave Period (sec)	Apparent Period (sec)	Current (ft/sec)	Current Angle CW From Platform North (degrees)
North	51.0	13.5	14.03	3.79	233.31
Northeast	61.2	13.5	14.35	3.87	234.58
East	68.0	13.5	14.20	3.86	236.55
Southeast	64.6	13.5	13.68	3.74	237.60
South	57.8	13.5	13.01	3.62	237.00
Southwest	47.6	13.5	12.65	3.58	235.35
West	47.6	13.5	12.85	3.61	233.84
Northwest	47.6	13.5	13.37	3.69	233.06

Table A.1.1-1
Seastate Parameters

Loading From Platform Direction	Jacket Lateral Wave and Current Load (kips)	Deck Lateral Wave and Current Load (kips)	Total Lateral Load (kips)	Wave Crest Elevation (feet)
North	1710	–	1710	30.3
Northeast	2732	48	2780	37.8
East	3152	78	3230	43.0
Southeast	2313	47	2360	40.4
South	1301	9	1310	35.2
Southwest	520	–	520	28.0
West	600	–	600	27.9
Northwest	1030	–	1030	27.9

Table A.1.7–1
Platform Lateral Loads

Linear 3–D Model with Non–Linear Pile Foundation

Lateral load level for first unity check = 1.0   1893 kips

Load Step	Lateral Displacement at Deck Level (+)43' at South East Leg feet	Lateral Loads kips	Element Failures	Component Failure Mode	Remarks
1	0.29	344			
2	0.45	516			
3	0.51	567			
4	0.63	688			
5	0.82	860			
6	1.03	1032			
7	1.26	1205			
8	1.50	1376			
9	1.78	1549			
10	2.09	1721			
11	2.44	1893	Row 2, Lev 3 Row 2, Lev 4 Row 2, Lev 8 Row 2, Lev 5 Row 1, Lev 6 Row 1, Lev 5 Row 1, Lev 4 Row 2, Lev 6 Row 1, Lev 3	Joint Failure K–Braces	
12	2.82	2065			
13	3.33	2330		Pile Failure	

Table A.4.1–1
Linear Results for East Storm Direction

Nonlinear 3–D Model with Joint Effects

Lateral load level for first member yield 1291 kips

Load Step	Lateral Displacement at Deck Level (+)43' at South East Leg feet	Lateral Loads kips	Element Failures	Component Failure Mode	Remarks
1	−0.003	Dead			
2	0.259	323			
3	0.540	646			
4	0.857	968			
5	1.035	1130			
6	1.224	1291	1922, 1940 19313	Initial yield hinges Pile/leg contact	Lev 1 Planframe Leg A2
7	1.425	1453	19113	Pile/leg contact	Leg B2
8	1.637	1614	1923, 1927, 1928, 1936 19513, 19713	Initial yield hinges Pile/leg contact	Lev 1 Planframe Legs A1 & B1
9	1.857	1776	3251 1921, 1927	Joint comp yield Initial yield hinges	Lev 3, Row 2 K–Brace Lev 1 Planframe
10	2.093	1937	3252 1922	Joint tens yield Full plastic hinge	Lev 3, Row 2 K–Brace Lev 1 Planframe
11	2.193	2002	2251, 2252, 3652	Joint tens/comp yield	Lev 2, Row 2 K–Braces Lev 3, Row 1 K–Brace
12	2.302	2066	4251, 4252 1922, 1927, 1936 1945	Joint tens/comp yield Full plastic hinge Full plastic hinge Initial yield hinges	Lev 4, Row 2 K–Braces Lev 1 Planframe Lev 1 Planframe Lev 1 Planframe
13	2.421	2131	2651, 2652, 3651, 4651, 4652 5251, 5252 1936	Joint tens/comp yield Full plastic hinge	Lev 2, Row 1 K–Braces Lev 3, Row 1 K–Braces Lev 4, Row 1 K–Brace Lev 5, Row 2 K–Braces Lev 1 Planframe
14	2.590	2195	5651, 5652, 6251, 6252, 6651, 6652 1927	Joint tens/comp yield Full plastic hinge	Lev 5, Row 1 K–Braces Lev 6, Row 2 K–Braces Lev 6, Row 1 K–Braces Lev 1 Planframe
15	2.888	2260	1932	Initial yield hinges	Lev 1 Planframe
16	3.040	2292	6101	Initial yield hinges	Lev 6, Leg 1
17	3.197	2324	1931, 1939	Initial yield hinges	Lev 1 Planframe
18	3.278	2341	1251	Strut buckling	Lev 1, Row 2 K–Brace
19	3.438	2357			
20	3.657	2365	6105	Initial yield hinges	Lev 6, Leg 5
21	4.009	2373			
22	5.095	2381	6101 1926 25407, 25408		Soil axial friction capacity exceeded

Table A.4.2–1
Yield Event Summary – East Storm – Fully Nonlinear EDP Model

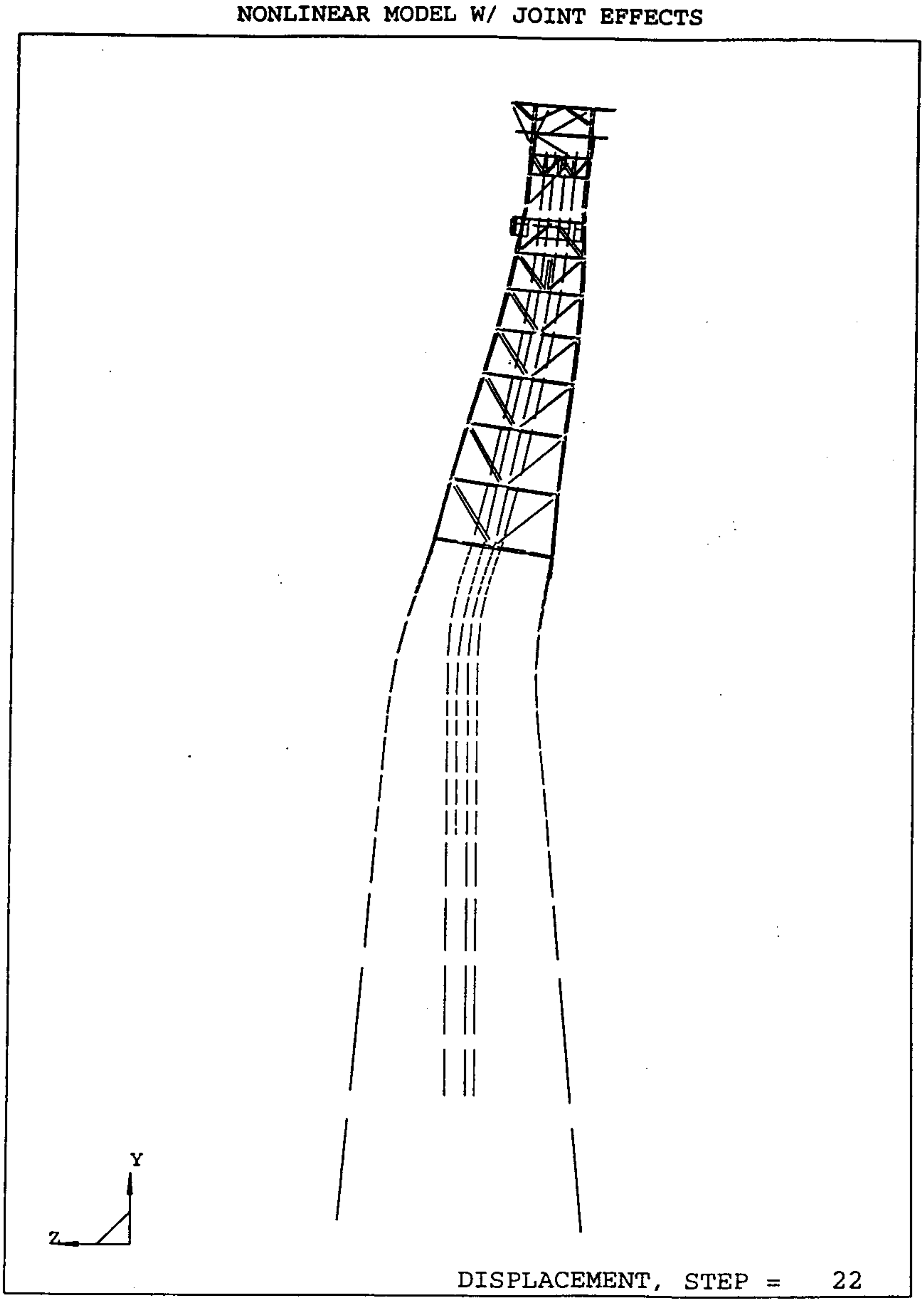


Figure A.4.2-2 East Storm Deformed Shape at RSR=0.74
- Fully Nonlinear EDP Model

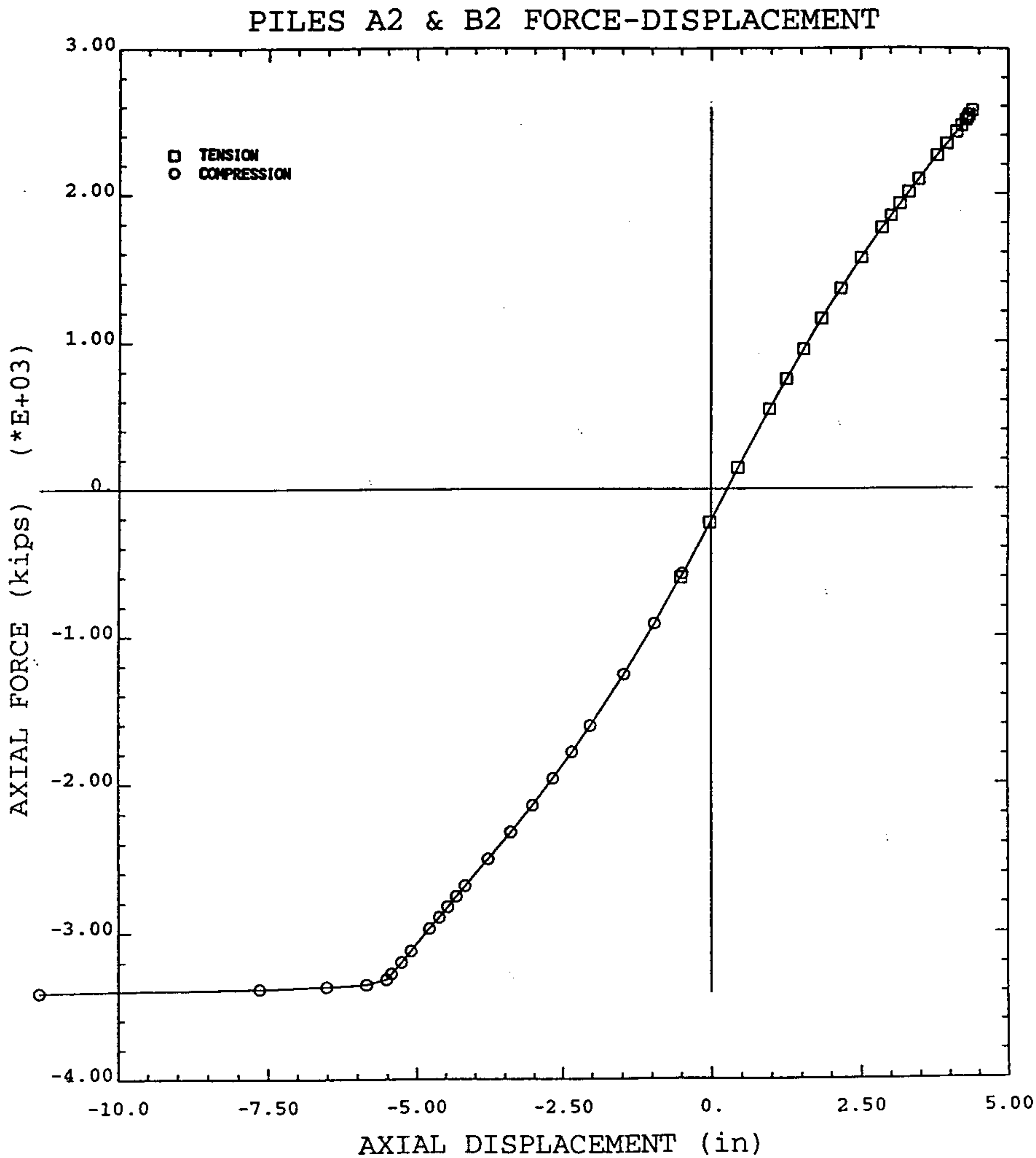


Figure A.4.2-3 Row 2 Tension and Compression Pile Axial Force Versus Axial Displacement East Storm - Fully Nonlinear EDP Model

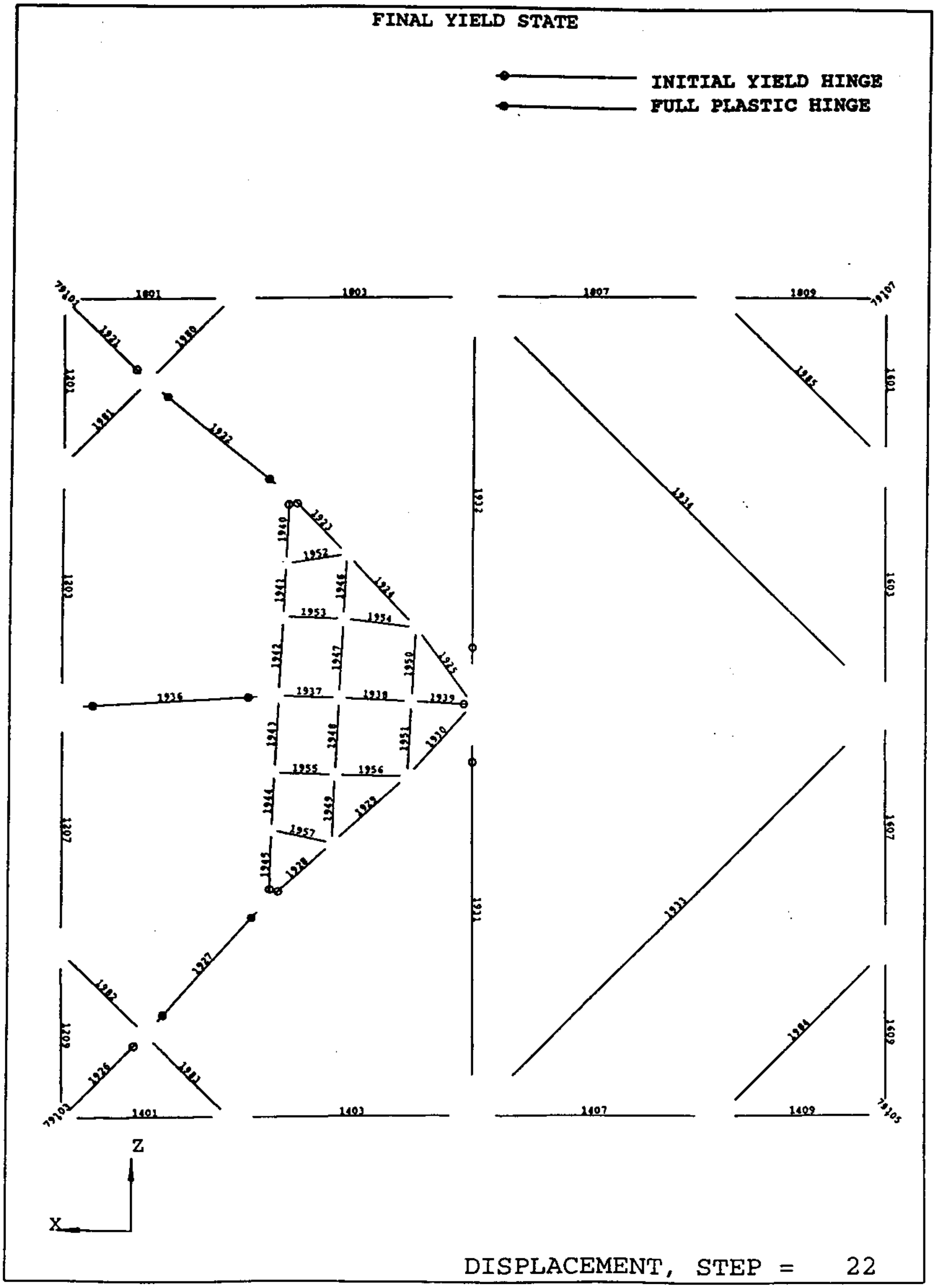


Figure A.4.2-4 Mudline Plan Frame Yield Events RSR=0.74 East Storm - Fully Nonlinear EDP Model

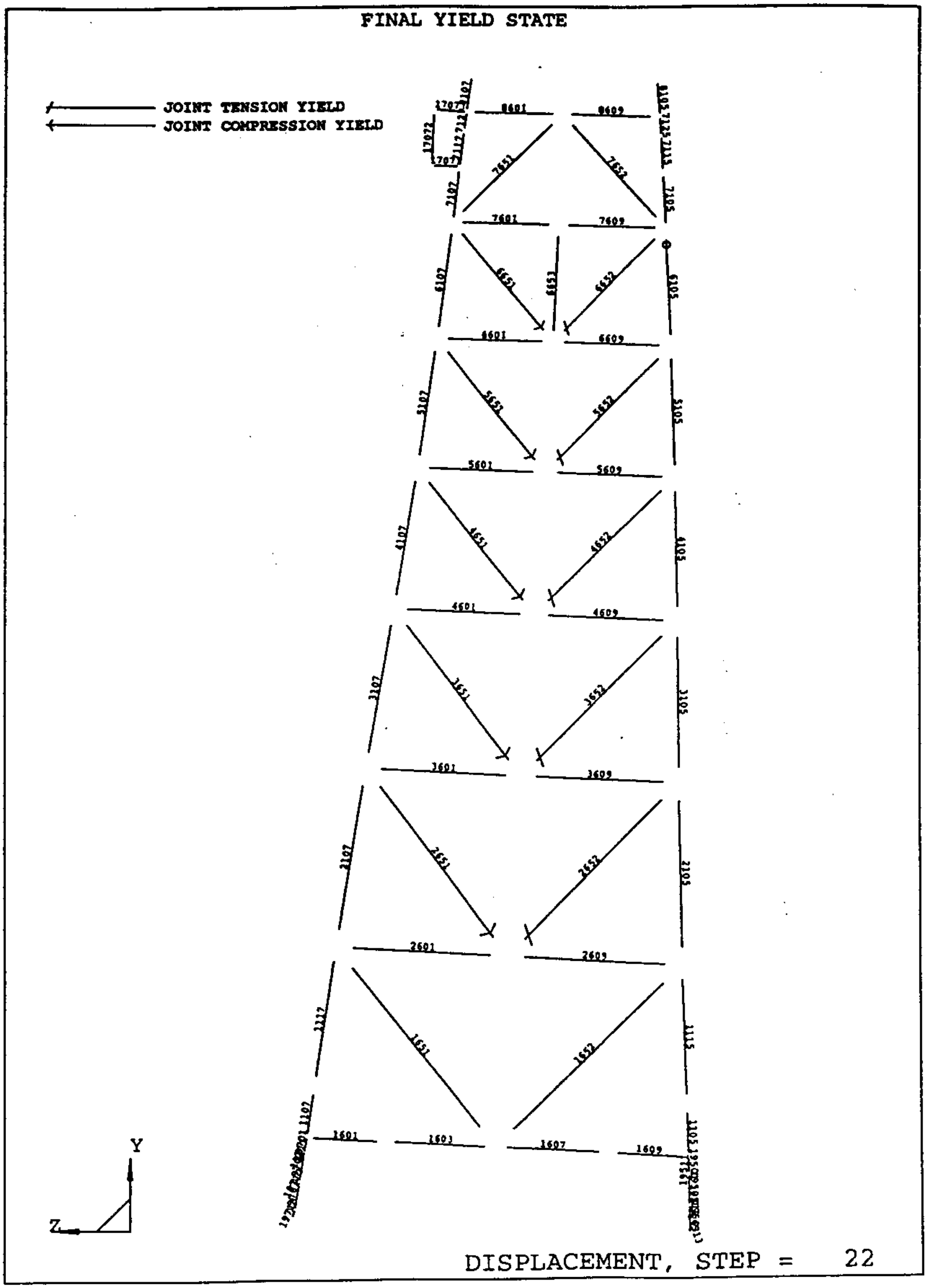


Figure A.4.2-5 Row 1 Yield Events at 0.74 East Storm RSR
- Fully Nonlinear EDP Model

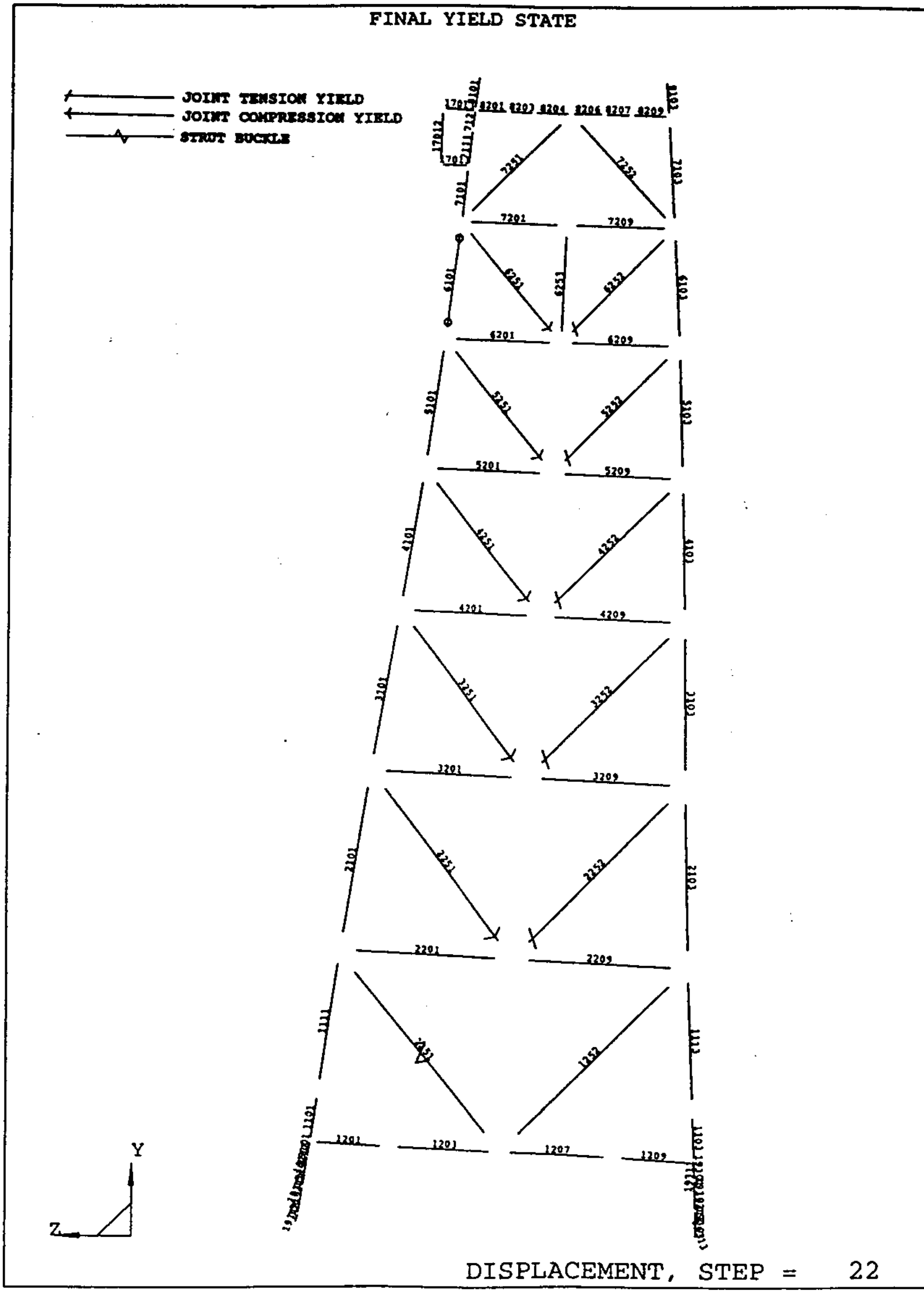


Figure A.4.2-6 Row 2 Yield Events at 0.74 East Storm RSR
- Fully Nonlinear EDP Model

PARTICIPANT "F"

SUMMARY

The benchmark platform was analyzed for ultimate strength using non-linear static pushover analysis methods. The platform was analyzed for loads from two directions--end-on/broadside and diagonal. Due to the platform orientation with respect to true north and the orientation of the API RP-2A wave criteria, the same wave height was used to generate loading patterns for both analyses. The KARMA structural analysis program was used in all analyses.

The structural analysis model of the platform was a 3-D model and included full pile modeling and full non-linear pile-soil interaction, including P-Y, T-Z, and Q-Z soil stiffnesses. The lateral load resisting capacity of the conductor foundations was included in the model (P-Y stiffness only). However, conductors were not modeled as structural elements above the mudline and thus did not contribute to the strength of the jacket.

In the structural model, the strengths of the vertical K-brace members were limited to the load carrying capacity of the K-joints. Capacities for these joints were calculated using API RP-2A LRFD formulas with resistance factors of 1.00. Joint strength was modeled in the analyses by using non-linear truss elements with elastic-plastic material properties and reduced yield strength to limit member load carrying capacity to the calculated joint capacity.

The benchmark platform behaved quite differently when analyzed in the two directions. For the broadside/end-on loading, joint capacity controlled the platform ultimate strength. Numerous vertical K-joints reached their load carrying capacity before any members other members failed in buckling or bending. For the analysis in the platform diagonal direction, ultimate platform strength was governed by foundation capacity (formation of pile double hinges).

The benchmark platform as analyzed did not meet the Section 17 ultimate strength criteria for loading from the diagonal direction (i.e. ultimate load factor was less than 1.00 based on Section 17 ultimate strength loading criteria). However, the platform did meet Section 17 ultimate strength requirements for the broadside/end-on loading case.

ENVIRONMENTAL CRITERIA

The platform was analyzed for environmental loading from two directions, one broadside/end-on case and one diagonal case, illustrated in Figure 1. The broadside/end-on loading case used waves and winds toward true northwest (315 degrees clockwise wrt true north) which corresponds to toward platform west. The diagonal loading case was toward true west (270 degrees clockwise wrt true north) which corresponds to toward platform southwest.

Wave forces on jacket members were based on Stokes 5th order wave theory and Morison's equation. Wave loading coefficients are listed in Table 1. Current was assumed constant for the two analysis directions: 3.88 ft/sec at 280 degrees toward, clockwise wrt true north. Wind forces were calculated on the projected deck areas given by PMB using the API RP-2A wind force procedures. One hour sustained wind speeds were used in wind force calculations.

Wave and current forces on decks were hand calculated using the draft Section 17 procedure. Magnitudes of wave in deck forces are listed in Table 2. Wave in deck forces were based on wave crest kinematics from Stream Function wave theory (the software was convenient). Wave forces on jacket members were based on Stokes 5th order wave kinematics since Stream Function wave theory

Table 1 -- Hydrodynamic Loading Coefficients		
Parameter	Environmental loading direction	
	315 degrees wrt true north Toward platform west	270 degrees wrt true north Toward platform southwest
Above MHHW:		
Drag coefficient, Cd	0.65	0.65
Inertia coefficient, Cm	1.60	1.60
Below MHHW:		
Drag coefficient, Cd	1.05	1.05
Inertia coefficient, Cm	1.20	1.20
Marine growth (in)	1.50 in	1.50 in
Wave kinematics factor:	0.88	0.88
Current blockage factor:	0.85	0.80
Conductor shielding factor:	1.00	1.00

was not convenient for use in the ultimate strength analysis software. The effect of mixing these two wave theories for the structural forces is likely minimal.

The kinematics reduction factor was implemented by using "effective" drag and inertia coefficients in the structural analysis software to calculate wave forces. The effective drag coefficient was the desired drag coefficient times the kinematics reduction factor squared; the effective inertia coefficient was the desired inertia coefficient times the kinematics reduction factor. Current velocities were amplified (by 1 over kinematics reduction factor) to offset the reduced effective drag coefficient. This method (inappropriately) also reduces vertical drag and inertia forces on jacket members, but the overall effect is minimal for wave forces based on the wave crest.

Wave forces were captured by statically passing a series of regular waves through the platform and capturing the wave force profile at the time of maximum total wave force on the platform. The two wave force patterns for the two analysis directions were then combined with the hand calculated wind and wave in deck forces and this force pattern was linearly increased to "push over" the platform.

3D MODEL GENERATION

The program KARMA was used for the non-linear static pushover ultimate strength analysis. The model was a full 3 dimensional model of the platform, including piles and non-linear spring elements to model non-linear soil behavior. Figure 2 illustrates the structural analysis computer model of the platform. A simplified model of the deck was used in the model, as shown in the Figure. Secondary conductor support framing at the horizontal jacket framing elevations was generally not modeled. Conductors, risers, boat landings, and boat bumpers were modeled using wave load only non-structural members. However, conductors below the mudline were modeled using structural elements to capture the conductor soil lateral resistance.

Table 3 summarizes the program KARMA element types were used in the ultimate strength of the platform.

Table 2 -- Data for Environmental Loads	
Number of approach directions:	2 (detailed below in this Table)
Approach used to determine wave/current in deck forces	Draft Section 17 procedure
Global 270 degrees (direction toward, clockwise wrt true north)	
Diagonal loading on platform, toward platform southwest	
Orientation wrt platform north:	Toward 225 degrees clockwise
Wave height:	68.0 ft
Wave period:	14.4 sec (incl. Doppler shift)
Current profile:	3.88 ft/sec @ 280 degrees, clockwise wrt true north, slab current profile
Storm surge:	3.0 ft
Wind speed @ 10 m above msl:	143 ft/sec in-line with wave direction
Magnitude of wave in deck load:	364 kips
Magnitude of wind load:	86.9 kips
Jacket wave force based on Section 17 ultimate strength criteria:	1783 kips
Global 315 degrees (direction toward, clockwise wrt true north)	
End-on/broadside loading on platform, toward platform west	
Orientation wrt platform north:	90 degrees clockwise
Wave height:	68.0 ft
Wave period:	14.2 sec (incl. Doppler shift)
Current profile:	3.88 ft/sec @ 280 degrees, clockwise wrt true north, slab current profile
Storm surge:	3.0 ft
Wind speed @ 10 m above msl:	143 ft/sec in-line with wave direction
Magnitude of wave in deck load:	309 kips
Magnitude of wind load:	64.9 kips
Jacket wave force based on Section 17 ultimate strength criteria:	1788 kips

References used to determine platform component strengths are summarized in Table 4. Member strengths for the analyses were based primarily on API RP-2A LRFD formulas. However, platform ultimate capacity was governed more by the strength of the vertical K-braces and thus not a great deal of effort was required in refining member strength estimates for buckling loads, etc. Joint capacity calculations are detailed in the following paragraphs. Where joint strength was less than member strength, the member was modeled using a non-linear truss element with material yield strength adjusted to the strength of the joint. The material was modeled as elastic-plastic, assuming the joint will maintain its load carrying capacity through relatively large displacements. K-factors for compression members were based on API RP-2A recommended values, with the exception of piles within the jacket legs. Since the gap between the piles and jacket legs was so small, pile buckling within the jacket legs is essentially restrained. Piles within jacket legs were given a very small effective length factor.

Joint strengths for the vertical K-joints were based on API RP-2A LRFD formulas using resistance factors of 1.00. For the K-joints, the factor Qf (which accounts for the presence of longitudinal

Table 3 -- KARMA Element Types Used in Analyses	
Deck members Conductors below the mudline	LBEM linear beam-column elements--linear material properties
Deck legs Jacket legs Piles inside jacket legs and piles below jacket Secondary horizontal jacket framing members	LANB non-linear large displacement beam-column elements--material non-linearity, geometric non-linearity, large displacement effects
Inclined members of vertical K-braces Horizontal members of vertical K-braces whose strength was controlled by joint capacities (T-joints at +10 ft and -157 ft)	NTRS non-linear truss elements, strength modified to model K-joint capacity--material non-linearity only to model joint behavior
Horizontal members of vertical K-braces whose strength was governed by member (not joint) strength	ISTR post-buckling strut element--buckling and post-buckling behavior when necessary--post buckling behavior based on phenomenological model from tests.
Pile/soil interaction for jacket piles	PSAS pile/soil interaction elements, including P-Y, T-Z, and Q-Z stiffnesses
Pile/soil interaction for conductors	PSAS pile/soil interaction elements, using P-Y stiffness only
Conductors above mudline, risers, boat landing members, boat bumpers	WAVL wave load only non-structural members
Pile/jacket leg interface inside jacket legs	SHER shear transfer elements

forces in the chord) was assumed constant at 0.90. This simplification was made to reduce the number of iterations through the ultimate strength analyses. K-joint gap distances for each joint were calculated from the member centerline intersection distances shown on the platform drawings. The K-joint at the -27 ft elevation was assumed to have a 16.6 inch gap distance, since the T-joint also framing in at this node will not be able to carry significant load. This large gap significantly reduces the load carrying capacity of this joint.

The capacity of the T-joints framing in at the +10 ft elevation and at the -157 ft elevation were calculated also from API RP-2A LRFD formulas. Member strengths for these joints were reduced to joint capacity using non-linear truss elements.

SOFTWARE DESCRIPTION

Software used in the ultimate strength analyses was the program KARMA from ISEC, Inc. The program KARMA is a general purpose structural analysis program for the linear and non-linear static and dynamic (time domain) analysis of structures. The program has several non-linear element types suitable for ultimate strength analysis of tubular space frame structures.

The program KARMA includes an automatic load stepping routine which was used for the non-linear static pushover analyses for this benchmarking study. The routine automatically reduces or reverses the applied load step factor as platform members fail and the platform stiffness softens. The program includes a wave loading routine which was used to generate static wave load profiles for the pushover analyses.

Table 4 -- Data for Member Capacity Estimation	
Material yield strength:	42 ksi
Member capacity estimates:**	
Braces	API RP-2A LRFD formulas with resistance factors = 1.00
Legs/piles	API RP-2A LRFD formulas with resistance factors = 1.00
Piles	API RP-2A WSD 20th edition
Joint capacity estimates:**	API RP-2A LRFD formulas with resistance factors = 1.00
Soil spring generation	API RP-2A WSD 20th edition
**Joint capacity controlled platform ultimate strength	

ULTIMATE STRENGTH ANALYSES

As outlined above, two ultimate strength analyses were performed. Considering the orientation of the platform and the symmetric nature of the 4-pile platform framing pattern, these two load cases were adequate to determine platform ultimate strength. The results of these analyses are described below and in several figures and tables attached to this report.

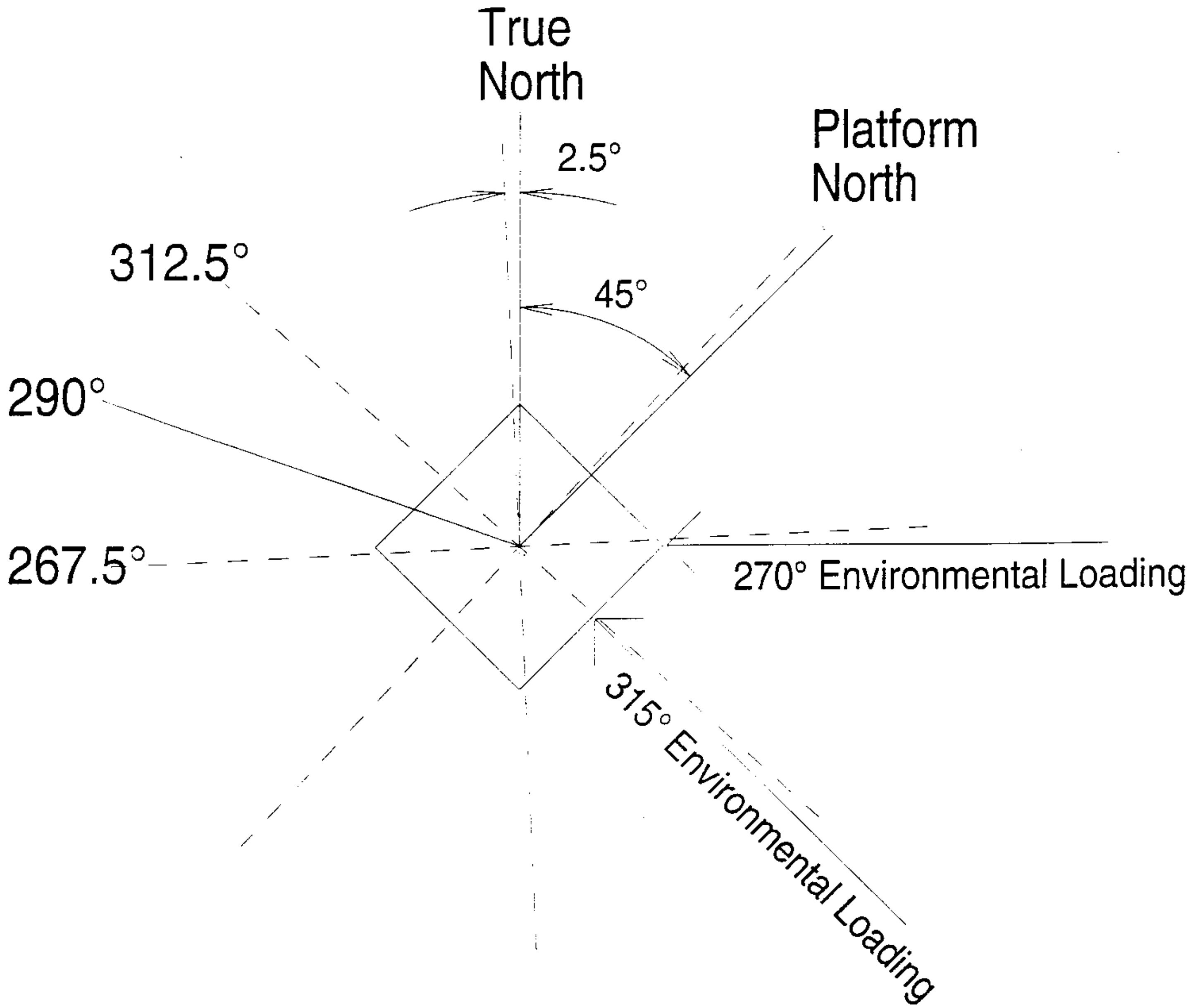
For the broadside/end-on loading case with environmental loading toward platform west, the ultimate capacity of the platform was determined to be 2545 kips. Based on API RP-2A 100 year reference level loads of 1680 kips, the platform RSR is 1.51. Figure 3 shows the load-displacement plot for the platform analysis. Figure 4 shows the same plot with component failures noted. The Figure 3 and 4 data are presented numerically in Table 4.

Figure 5 shows on a sketch of the platform the sequence of member failures. As shown in the figure, platform capacity was controlled by the strength of the vertical K-joints. At several load steps, multiple joints failed simultaneously. Figure 6 shows the displaced shape of the benchmark platform, with displacements magnified 15 times, at the end of the analysis.

For the diagonal loading direction analysis, platform strength was governed by the strength of the foundation. Ultimate platform strength for this direction was determined from analysis to be 1937 kips. Using an API RP-2A 100 year reference load of 1713 kips, the calculated RSR for this direction is 1.13. The platform did not pass the Section 17 ultimate strength criteria for this loading direction (load factor on Section 17 loads at ultimate platform strength was less than 1.00).

Figure 7 illustrates the load-displacement behavior of the platform for the diagonal loading analysis. No component failures were experienced before the platform foundation reached its ultimate strength, as illustrated in Figure 8. The data plotted in Figures 7 and 8 are listed in Table 5. Figure 9 illustrates the displaced shape of the platform at ultimate strength, with displacements magnified 15 times.

Figure 1 -- Wave Directions for Analysis and Platform Orientation



Maximum API RP-2A wave height is toward 290° wrt true north. This maximum wave height applies to a range of +/- 22.5°. The platform orientation of 45° wrt true north places both the platform broadside/end-on and diagonal directions approximately within this range. Analyses were performed using the same maximum wave criteria for "true broadside/end-on" and "true diagonal" directions, i.e. waves toward 270 ° and 315 ° wrt true north.

Wave height for both analyses was 68.0 ft.

Current direction was toward 280° wrt true north.

Figure 2 – KARMA Structural Analysis Model

(full length of piles not shown)

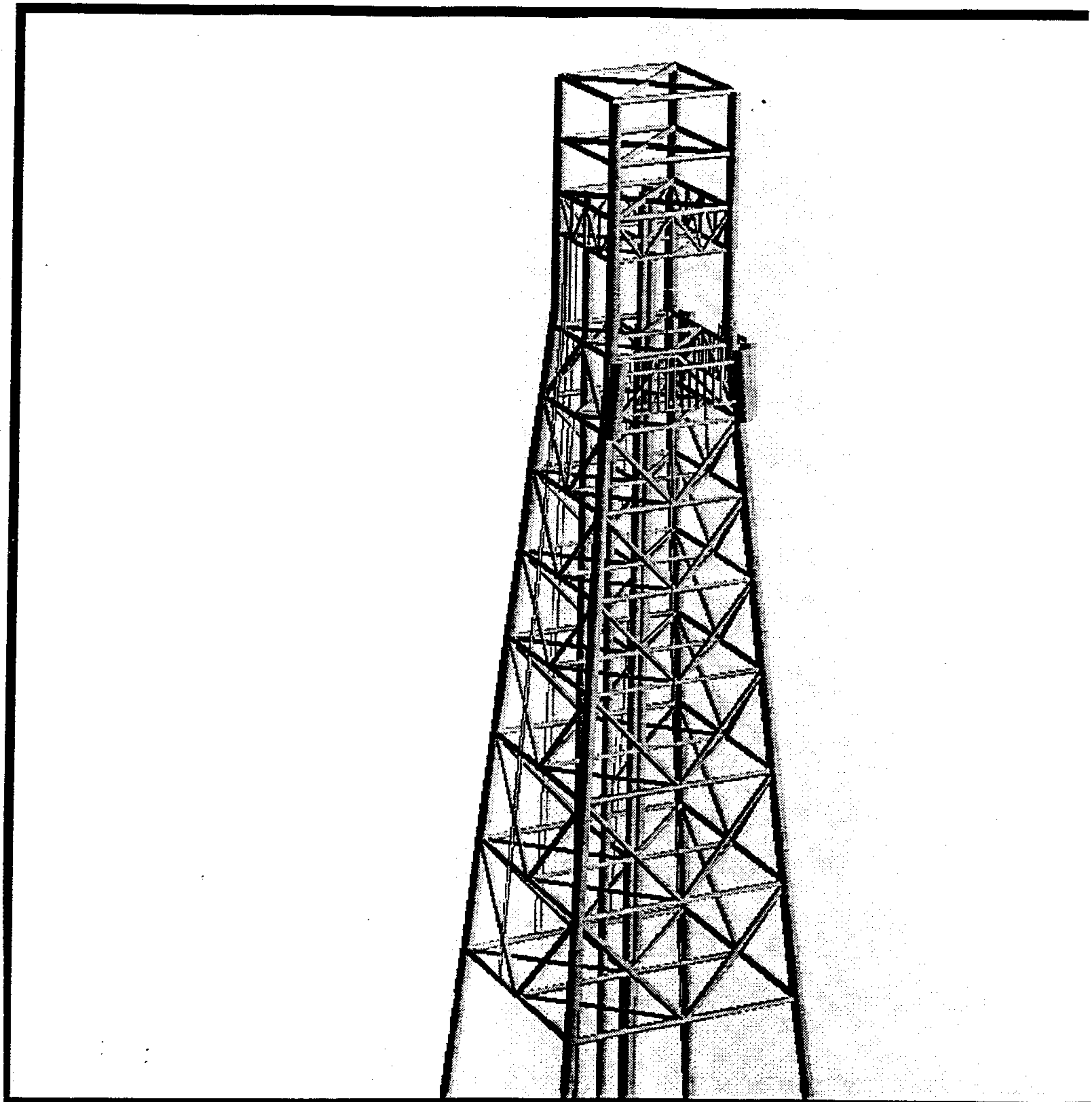
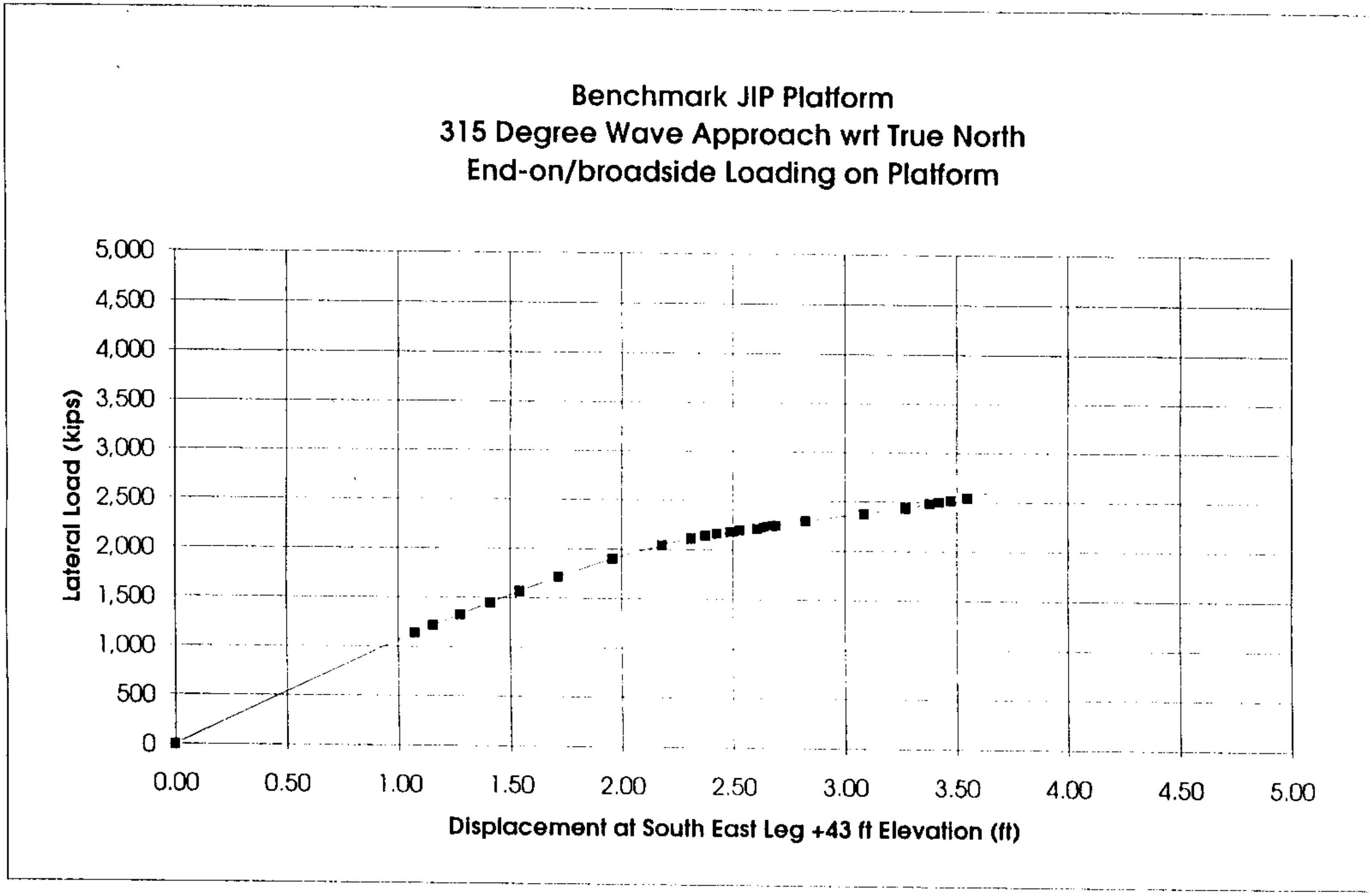


Figure 3 -- Results of End-on/Broadside Analysis



Reference level load:	1680	kips	(Based on API RP-2A 100 yr criteria)
Load level at first member failure:	1905	kips	(Vertical K-joint failures)
Ultimate capacity:	2545	kips	
Reserve strength ratio:	1.51		=Ultimate capacity / reference level load
Platform failure mode:	Joint failure in vertical K-joints, followed by jacket leg portal frame collapse		

Figure 4 -- Results of End-on/Broadside Analysis with Component Failures Noted

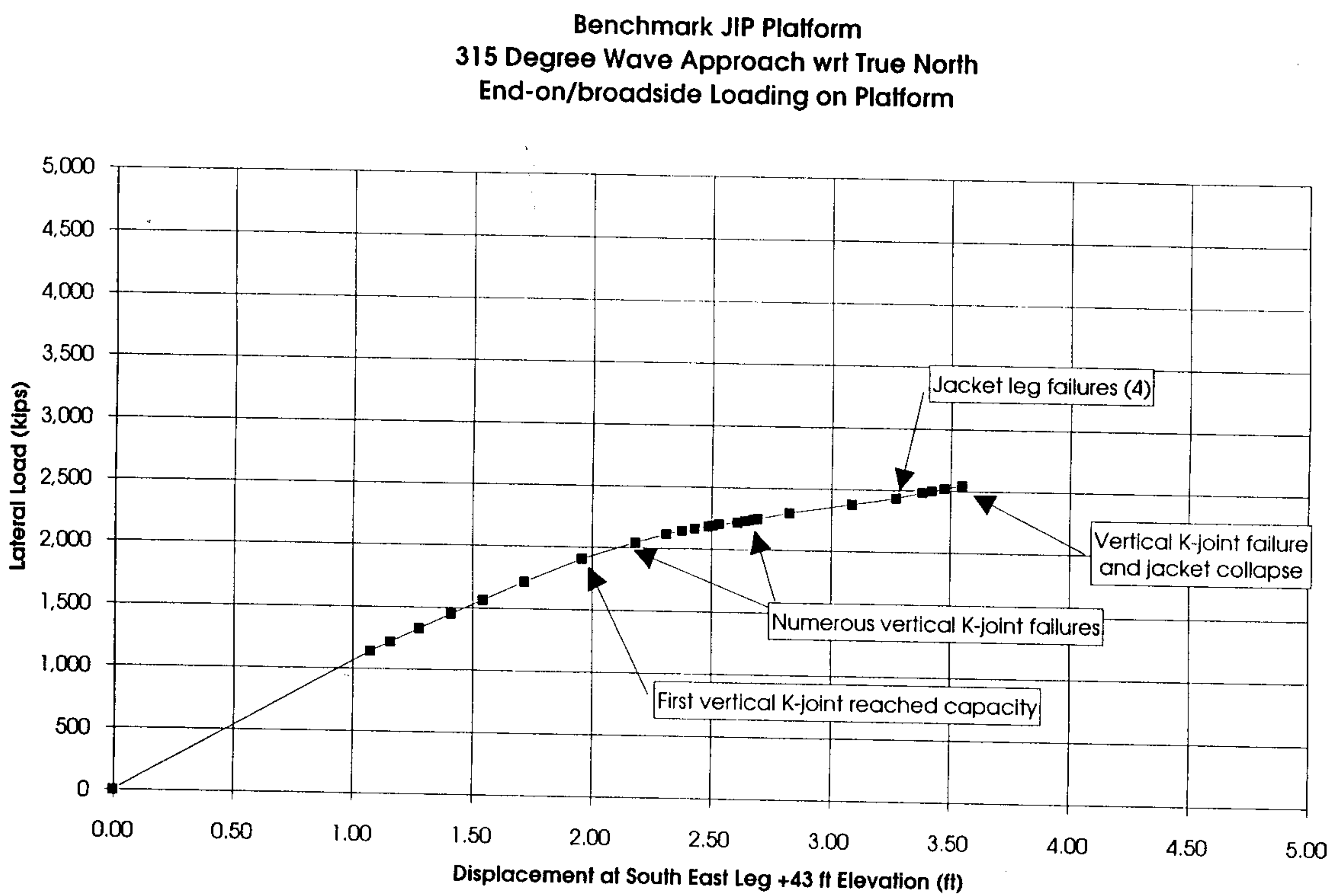


Figure 5 -- Illustration of Failure Sequence from End-on/Broadside Analysis

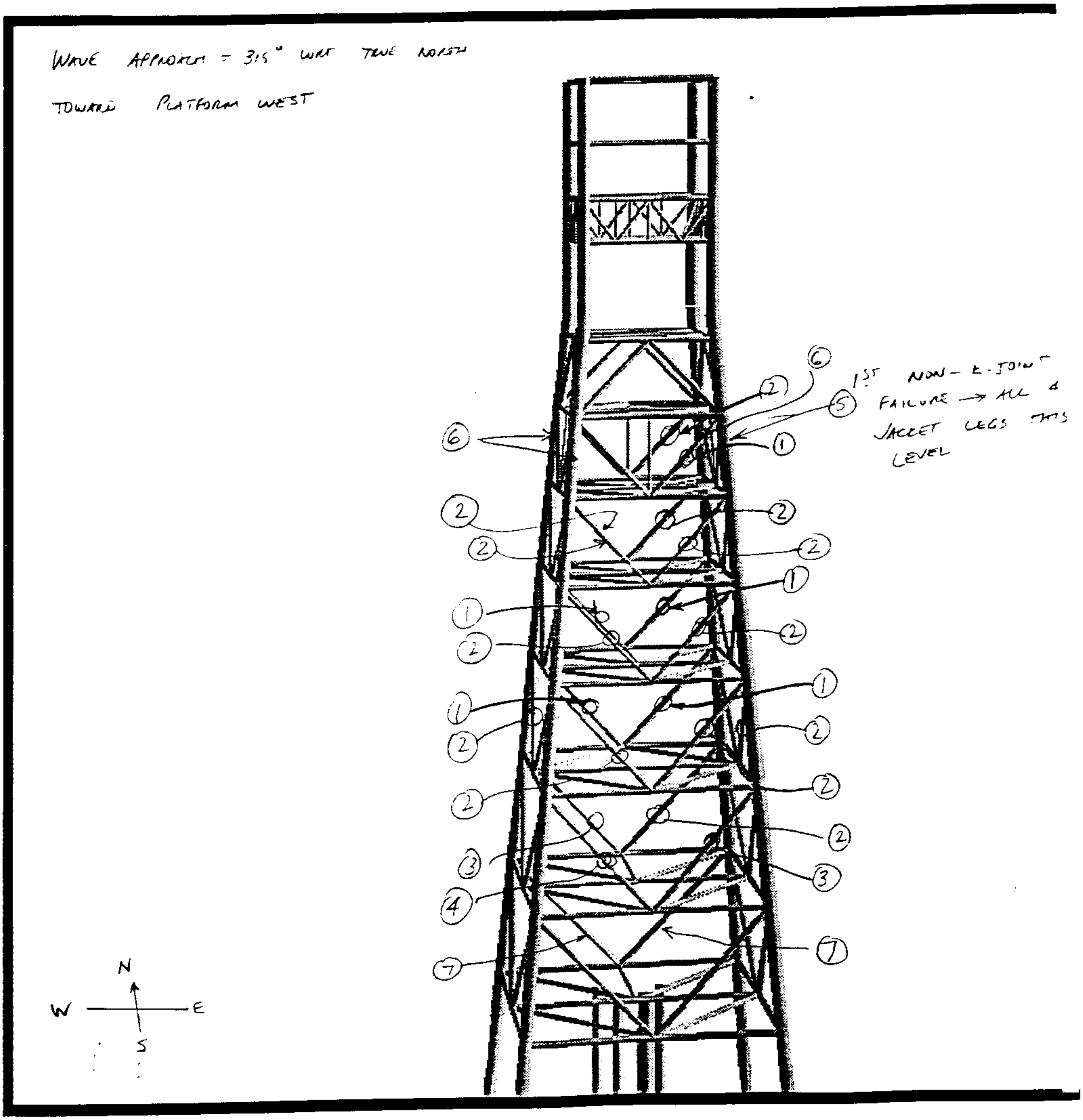


Figure 6 -- Displaced Platform Shape for End-on/Broadside Analysis

(displacements magnified 15 times)

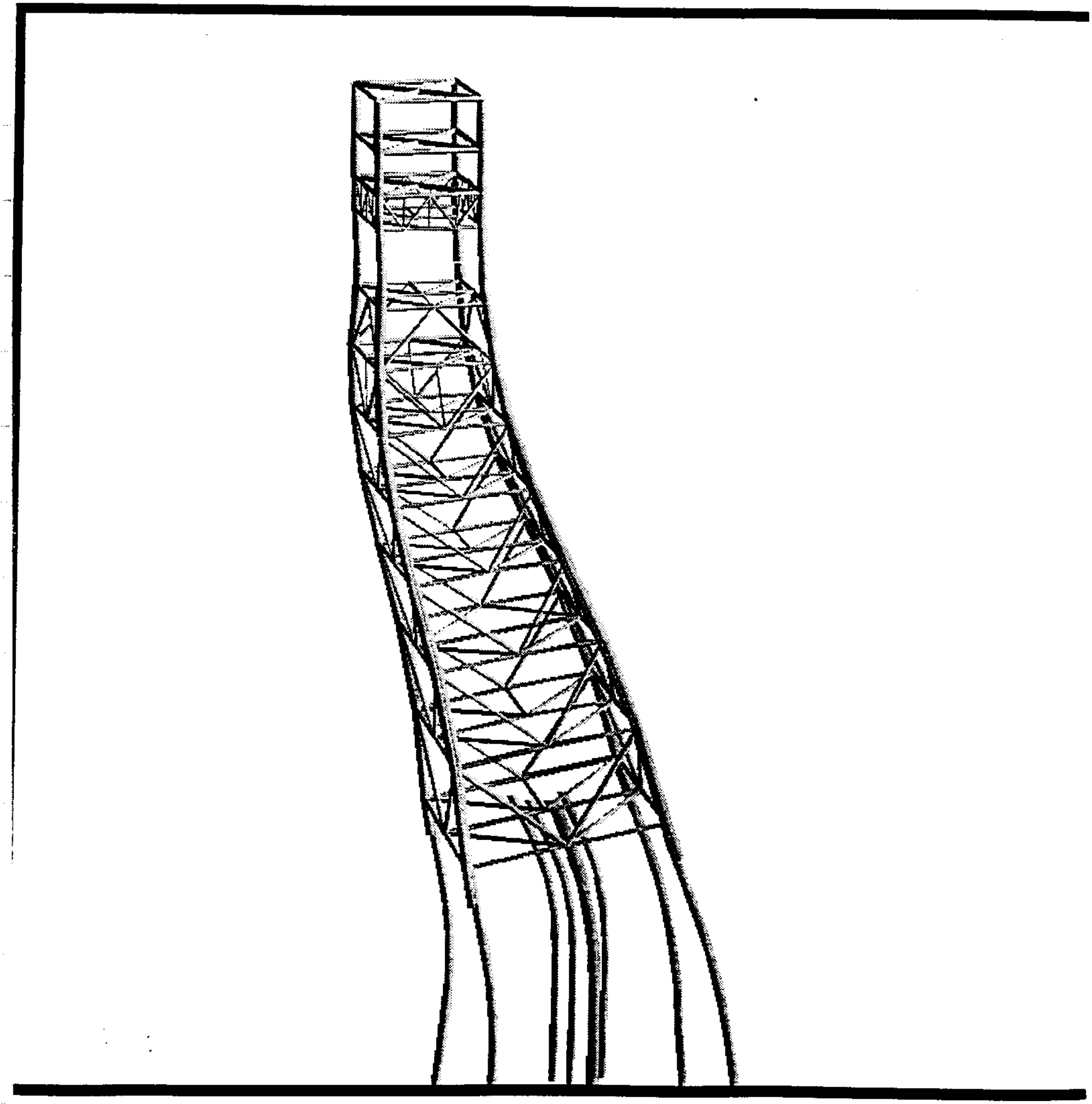
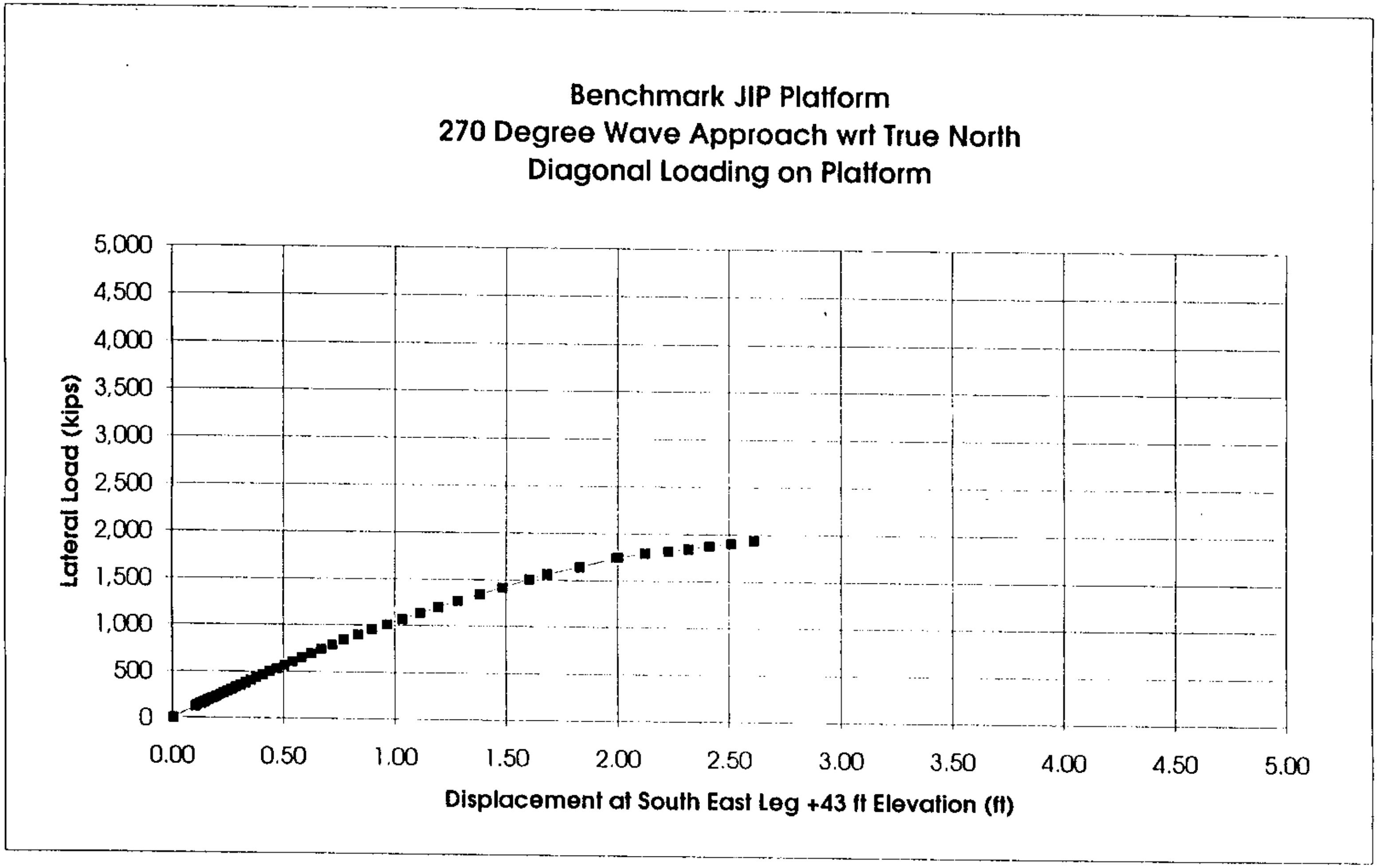


Figure 7 -- Results of Diagonal Analysis



Reference level load:	1713	kips	(Based on API RP-2A 100 yr criteria)
Load level at first member failure:	1937	kips	(Pile double hinging)
Ultimate capacity:	1937	kips	
Reserve strength ratio:	1.13		=Ultimate capacity / reference level load
Platform failure mode:	Foundation failure -- pile double hinging		

Figure 8 -- Results of Diagonal Analysis with Component Failures Noted

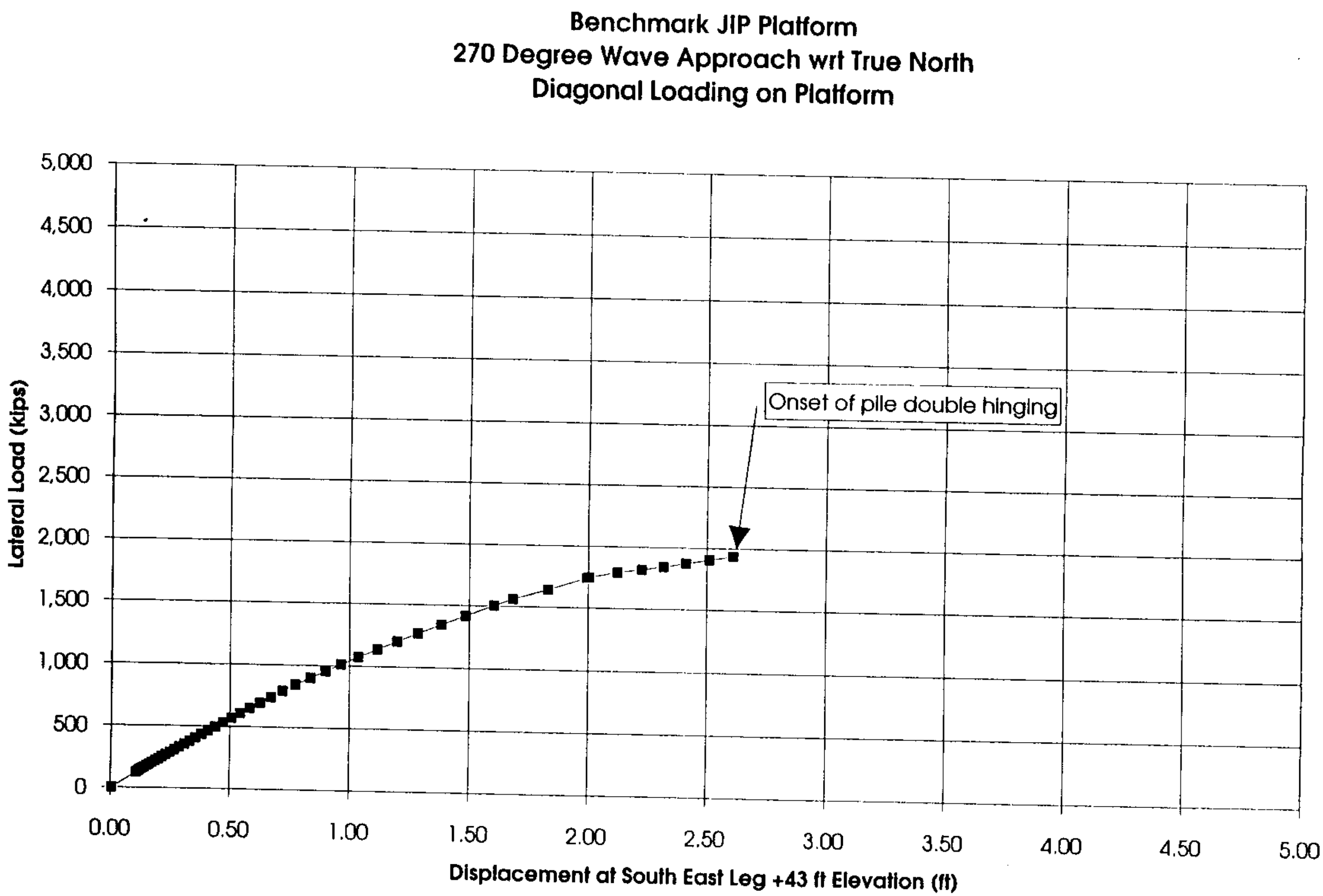


Figure 9-- Displaced Platform Shape for Diagonal Analysis

(displacements magnified 15 times)

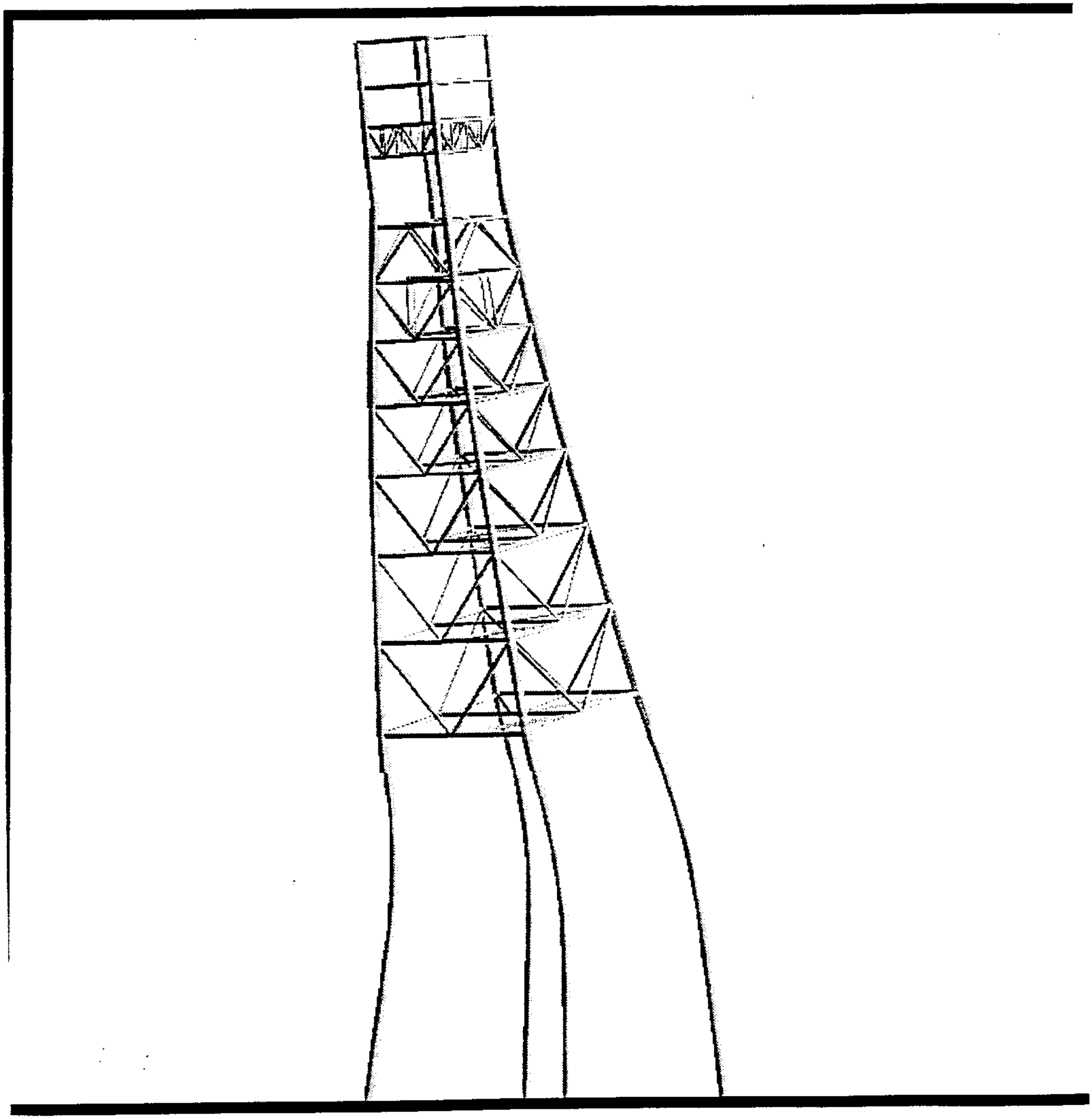


Table 5 -- End-on/Broadside Analysis Load-Displacement Data

Load-Displacement Data

315 Degree Wave Approach
(wrt True North)

Broadside loading on platform
Toward platform west

Load Step	Displ. SouthEast Leg Elev +43 ft (ft)	Lateral Load (kips)	Element Failures	Component Failure Mode
1	0.000	0		
9	1.073	1,142		
10	1.156	1,218		
11	1.277	1,330		
12	1.411	1,451		
13	1.544	1,567		
14	1.716	1,716		
15	1.958	1,905	5 Vertical K-joints noted (1)	Joint capacity
16	2.181	2,044	12 Vertical K-joints noted (2)	Joint capacity
17	2.311	2,115	2 Vertical K-joints noted (3)	Joint capacity
18	2.377	2,144		
19	2.429	2,164		
20	2.487	2,182		
21	2.505	2,191		
22	2.531	2,202		
23	2.608	2,216		
24	2.640	2,229	1 Vertical K-joint noted (4)	Joint capacity
25	2.655	2,235		
26	2.676	2,244		
27	2.689	2,250		
28	2.692	2,251		
29	2.825	2,301		
30	3.086	2,378		
31	3.271	2,434	First jacket leg failure (5)	Bending
32	3.381	2,483	3 jacket leg failures (6)	Bending
33	3.421	2,498	2 Vertical K-joints noted (7)	Joint capacity
34	3.474	2,517		
35	3.549	2,545	Jacket collapse	Portal frame action

Table 6 -- Diagonal Analysis Load-Displacement Data

Load-Displacement Data

270 Degree Wave Approach
(wrt True North)

Diagonal loading on platform
Toward platform southwest

Load Step	Displ. SouthEast Leg Elev +43 ft (ft)	Lateral Load (kips)	Element Failures	Component Failure Mode
1	0.00	0		
9	0.11	126		
10	0.11	135		
11	0.12	145		
12	0.13	155		
13	0.14	166		
14	0.15	178		
15	0.16	191		
16	0.17	205		
17	0.19	219		
18	0.20	235		
19	0.22	252		
20	0.23	269		
21	0.25	288		
22	0.27	309		
23	0.29	330		
24	0.31	354		
25	0.33	378		
26	0.35	405		
27	0.38	434		
28	0.41	465		
29	0.44	497		
30	0.47	532		
31	0.51	569		
32	0.54	608		
33	0.58	649		
34	0.63	693		
35	0.67	741		
36	0.72	790		
37	0.77	843		
38	0.84	898		
39	0.90	954		
40	0.97	1,012		
41	1.04	1,073		
42	1.12	1,136		

Load-Displacement Data (continued ...)

270 Degree Wave Approach
(wrt True North)

Diagonal loading on platform
Toward platform southwest

Load Step	Displ. SouthEast Leg Elev +43 ft (ft)	Lateral Load (kips)	Element Failures	Component Failure Mode
43	1.20	1,201		
44	1.29	1,270		
45	1.38	1,343		
46	1.48	1,416		
47	1.61	1,505		
48	1.68	1,561		
49	1.83	1,643		
50	1.99	1,742		
51	2.00	1,745		
52	2.00	1,748		
53	2.12	1,791		
54	2.23	1,819		
55	2.32	1,843		
56	2.41	1,874		
57	2.51	1,905		
58	2.61	1,937	Pile failures	Pile double hinging

PARTICIPANT "G"

TRIAL APPLICATION OF THE DRAFT API RP 2A-WSD PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

BENCHMARK DOCUMENT

TABLE OF CONTENTS

INTRODUCTION

As part of the trial application of the draft API RP 2A-WSD Draft Section 17 Procedures for Assessment of Existing Platforms Joint Industry Project (JIP), design level and ultimate strength analyses were performed on a benchmark platform. The objectives were 1) to identify variations in platform ultimate strength obtained by the use of different methods and software and 2) to identify any problems in interpretation or in the technical applicability of the draft Section 17 guidelines. The benchmark platform was installed in 1970 in 157 ft. water depth in the Ship Shoal area of the Gulf of Mexico. It has 4-legs with K-braces in the vertical frames and ungrouted leg-pile annuli. The platform has 4-production wells and a quarters facility. The platform was categorized as a "manned-evacuated, significant environmental impact" platform. This trial application is based on the following:

1. *API RP 2A-WSD 20th Edition, Draft Section 17, Assessment of Existing Platforms,* dated April 29, 1994.

2. *API RP 2A-WSD Draft Section 17.0 - Balloted Changes,* Letter from K. A. Digre to API Task Group 92-5 dated June 29, 1994.

3. *API RP 2A-WSD, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms - Working Stress Design,* Twentieth Edition, July 1, 1993.

4. *Trial Application of the Draft API RP 2A-WSD Procedure for Assessment of Existing Platforms - Benchmark Basis Document,* PMB Engineering, Inc., dated February 1, 1994.

5. *Trial JIP - Benchmark Analysis - Revision 1,* Letter from F. J. Puskar to Trial Application Participants dated March 25, 1994.

6. *Trial JIP - Benchmark Analysis - Revision 2,* Letter from R. K. Aggarwal to Trial Application Participants dated April 12, 1994.

7. *Trial JIP - Benchmark Analysis - Revision 3,* Letter from R. K. Aggarwal to Trial Application Participants dated April 20, 1994.

1.0 SUMMARY

This section contains a brief note introducing the software, element types, and key computer modeling techniques used to determine the ultimate strength of the benchmark platform. Primary results form the Ultimate Strength analysis are also presented.

a) Software
The Design Level analysis and the Ultimate Strength linear analysis were performed using program "X". "X" is an in-house comprehensive structural analysis program especially suited to performing linear elastic analyses of large space frames. PILE"X", a nonlinear soil-pile-structure interaction analysis capability available in "X", was used to model the

piles and the soil. The wave, current, and gravity forces on the platform were generated by "Y", an in-house preprocessor for the "X" program. KARMA (INTRA) was used to perform the ultimate strength nonlinear analyses for detailed assessment of ultimate capacity. KARMA (INTRA) is a 3-D, inelastic, nonlinear response analysis program, based on finite element formulations, for offshore structures subjected to environmental loads.

b) Modeling techniques for Ultimate Strength nonlinear analysis

The 3-D analytical model of the platform included all primary and secondary structural members in the jacket and deck. For wave load and mass calculations, the platform appurtenances (boat landing, risers, stairs, conductors, etc.) were modeled as nonstructural elements. The contribution of the conductors to the foundation resistance of the platform was not considered. In addition, the ultimate strength analysis did not consider the strength of the jacket joints.

The legs, piles, and horizontals were expected to yield during the ultimate strength nonlinear analysis, primarily due to high bending stresses. Therefore, they were modeled as beam-column elements. The diagonals were modeled as strut elements because they were expected to undergo axial yielding or buckling. The deck members were expected to perform elastically, thus, were modeled as linear beam elements. The "wishbones" connecting the piles to the jacket were modeled as shear elements.

c) Ultimate Shear Capacity and Reserve Strength Ratio

The ultimate capacity was approximately 1600 kips for the four approach directions analyzed. The reserve strength ratios, based on API RP 2A-WSD, 20th edition loading, ranged from a minimum of 0.57 for the West approach to a maximum of 1.21 for the North approach. The maximum base shear on the platform calculated with the Section 17 Ultimate Strength metocean criteria was 3430 kips. The platform, therefore, fails Section 17 Ultimate Strength Analysis in its intact condition for the assessment category selected.

PART A: BENCHMARK ANALYSIS

A.1 Environmental Criteria

A.1.1 Design Level Analysis

Since this platform is categorized as "Significant Environmental Impact/Manned-Evacuated", the metocean loading criteria is in accordance with Section 17.6.2a of the Draft guidelines. The design level wave height, storm tide, current speed, wave period, and wind speed were found in Table 17.6.2-1 of the Draft guidelines. The design level wave height and current speed were to be omnidirectional. However, the wave height and current speed versus direction exceeded that required by the 20th edition for new designs for several approach directions. In these directions the 20th edition criteria governed. Table 1 contains the environmental criteria as defined by the 20th edition and the draft guidelines. Figure 1 shows the wave approach directions, wave heights, current velocities, and current directions used in the analysis. The wind was assumed in the same direction as the wave for all approach directions.

A.1.2 Ultimate Strength Analysis

The environmental criteria for the ultimate strength analysis was developed in the same manner as for the design level analysis. The ultimate strength wave height, storm tide, current speed, wave period, and wind speed were found in Table 17.6.2-1 of the Draft guidelines. The ultimate strength wave and current directionality were found in the 20th edition as instructed by Table 17.6.2-1. The 20th edition recommends that the wave height vary according to direction as shown in Figure 2.3.4-4 and the current remain constant at 270 degrees clockwise from "true" North, as shown in Figure 2.3.4-5. The current speed for an approach direction other than 270 degrees clockwise from North was found by computing the in-line component of the current. Those components found to be less than 0.2 knots were set equal to 0.2 knots as per Section 2.3.4c.4. Figure 2 shows the wave approach directions, wave heights, current velocities, and current directions used in the analysis. Other environmental criteria used in the ultimate strength analysis are presented in Table 1. For information, Table 2 presents the ultimate strength environmental loading data in the format requested for the "Benchmark Document". The wind direction was assumed in the same direction as the wave.

In accordance with the Benchmark Analysis workscope, reserve strength ratio (RSR) calculations were to be based on API RP 2A-WSD, 20th edition loading (i.e. the reference level load, Sref, in the RSR denominator should be based on 20th edition 100-year loading). Table 1 and Figure 3 provide the environmental criteria used for calculating Sref for each of the wave approach directions considered.

A.2 3-D Model Generation

The 3-D model was generated using "X". A geometric representation of the 3-D model is shown in Appendix A. A design level analysis was completed to ensure accurate modeling of the 3-D structure and to identify members that would most likely yield during the ultimate strength analysis. The load-resistance behavior of the soil was modeled by discrete axial and lateral springs distributed along the length of the pile. Explicit P-Y curves were generated by PILE"X" to model the response of soft clay. Consistent with standard industry practices, cyclic pile criteria were used. Five soil layers were defined from interpretation of the soil information given. The software used allowed only one type of soil for a given pile, therefore, "equivalent" soil properties for the intermediate sand layer were found by linear interpolation of the surrounding soil layers. Default T-Z and Q-Z curves were modeled after Coyle and Reese.

The draft guideline was followed to determine the environmental load criteria as described in Section A.1 of this document. To ensure accurate modeling of the nonstructural members, these members were removed by groups to determine the amount of force obtained from the environmental loads. By examination of Figure 4, it is apparent that the magnitude of the loads caused by each nonstructural group is reasonable. This verified "X" model was then translated into an INTRA model in order to complete the ultimate strength analysis.

A.3 Software Description

"X" is a comprehensive structural analysis program especially suited to perform linear elastic analyses of large space frames. "X" is a versatile tool for the design engineer and can be effectively used for the analysis and design of offshore platforms composed of beam elements. The beam elements (or members) are considered to be long in comparison to their cross-sectional dimensions. The joints of the structure are points of intersection of the members, as well as points of support and free ends of members. "X" uses the stiffness matrix method in an approach that operates on substructures in a forward reduction, backward substitution procedure.

PILE"X" is the nonlinear soil-pile structure interaction analysis used by "X". Structure foundations can be modeled by single piles imbedded in soil strata of clay or sand. The overall structural solution is obtained after a stepwise linear iteration procedure converges on conditions of compatibility and equilibrium at the pile-structure interface.

"Y" is a comprehensive computer program that automates the preparation of the wind, wave, current, dead, and buoyancy load data required for the static space frame analysis of a wide range of marine structures. "Y" moves a rigid wave, with or without current, through the structure in order to determine the wave position resulting in the maximum shear or the maximum overturning moment on the structure. The program can orient the wave in any direction with respect to the structure and can orient the current in a direction different from the wave direction. The user controls the wave and current directions and the iterative procedure employed in determining the maximum shear or overturning moment. This iterative procedure is controlled by defining the initial and final positions of the wave crest for a test interval and by defining the spacing between consecutive trial positions over the test interval. The load data for the selected maximum is produced in the card image format required by "X". Multiple design load conditions may be investigated by defining different wave and/or current characteristics and directions.

Wave and current forces per unit length, consisting of drag and inertia contributions as well as Froude-Krylov forces, are computed for several segments along each member in the wave. Forces are computed at an optimum number of intermediate points along the member, based on the difference in force between the member ends. Linearly varying distributed member loads, defined by the end and intermediate forces, are generated for each member. This load optimization feature ensures that "Y" will generate the minimum quantity of loading data required to accurately model the wave load distribution on each member. If the member intersects the wave surface or mudline, only the portion of the member in the wave is considered when calculating the forces. If the member ends are above the wave surface or below the mudline, no wave or current forces are calculated.

In addition to wave and current forces, "Y" is used to generate wind forces on the structure. Wind forces on the defined surface areas (modeled to represent the deck equipment, housing, etc.) were generated. The wind forces were produced by a constant wind velocity. These wind forces are added to the base shears and overturning moments calculated from the wave and current forces. The wind is always assumed to travel in the same direction as the wave.

"Y" was also used to generate the dead and buoyancy loads for all members in the structure except the deck members. Uniformly distributed loads over the member length are generated in the "X" format. A member may be considered to be flooded, unflooded, or weightless. The buoyancy of a member is calculated if the member is below the still water line. If a member intersects the still water line, then the portion of the member below the still water line is considered buoyant and the portion above is considered in air.

Nonstructural members and additional load carrying segments are modeled in "Y". This capability has been designed to simplify the structural model by elimination of the members that contribute environmental loads to the structure but do not contribute to its structural integrity.

The computer program KARMA(INTRA) was employed in the push-over analysis. KARMA is a three-dimensional, inelastic, nonlinear, static and dynamic finite element analysis program for offshore structures subjected to environmental loads. Static and dynamic analyses capabilities can be effectively used to evaluate ductility, failure modes or structural integrity of jacket type or deepwater compliant structures. Dynamic analyses are performed in the time domain or the frequency domain. Environmental loads may be wind, wave, earthquake, boat impact, or any generalized loading. The program can handle material, as well as geometric nonlinearities. KARMA has several built-in nonlinear solution schemes. These include the Newton-Raphson, modified Newton-Raphson, Constant Stiffness Path Dependent, Constant Stiffness Path Independent, Step-by-Step, Step-by-Step with equilibrium correction and others. For this analysis, a modified Newton-Raphson solution scheme was used. All of the loads in KARMA are translated from "Y" as nodal loads. The soil data is generated from PILE"X" data. The piles below the mudline are translated into KARMA from "X".

A.4 Force Generation

A.4.1 Gravity Loads

The dead and live loads for the deck were given in Appendix 2 of the Benchmark Basis Document. The live loads were applied as concentrated loads at the columns of each deck level. The dead load was given as a concentrated load, but was translated to a distributed load along the column rows for member force generation purposes. The dead load of the piles below the mudline was computed and applied at the mudline. The dead load of the jacket, as well as the buoyancy load of the platform, was computed by "Y" as uniformly distributed member loads. Table 3 contains a summary of all gravity loads.

A.4.2 Wave, Wind, and Current Loads

The environmental criteria, as described above, were used to generate the environmental forces. The wind loads were generated by supplying "Y" the projected areas of the deck and the wind shape coefficients as given in the Benchmark Basis Document. The maximum base shear and overturning moment calculated for each direction for the design level and the ultimate strength analyses are shown in Figures 5 and 6, respectively.

A.4.3 Wave/Current Deck Loads

The wave/current deck forces were calculated according to Section C17.6.2 in the Draft Section 17 guidelines. The silhouette area for this platform was defined as the area above the cellar deck bottom of steel at elevation +42.13 (i.e. wave/current deck forces were considered only when the wave crest elevation exceeded 42.13 ft.). The framing at elevation +33.00 and the trussing between elevation +33.00 and +42.13 were modeled with tubular members (per the drawings) and the forces were calculated using the wave load preprocessor program "Y". Table 4 shows the data used for wave/current deck force calculations. As shown, only one approach direction resulted in waves extending into the deck.

A.5 Analysis Checks

The analysis checks were performed in accordance with the data in Table 5.

A.5.1 Design Level Analysis

The design level analysis was performed for eight approach directions using "X". The design level analysis was performed to ensure the completeness and accuracy of the 3-D model and to predict an expected failure mechanism. Results of the member stress analysis showed that not all structural members are adequate for the Design Level Analysis metocean loading. The piles and segments of the jacket legs were found to be overstressed in several of the approach directions. These members experiencing unity checks greater than 1.0 are shown in Appendix B. Large pile-head deflections were encountered for all approach directions. These large deflections were the result of limited soil restraint (weak surface soils with low pile stiffness). Since the analysis indicated inadequate lateral stiffness of the piles (bending overstress), axial pile capacity was not assessed. In addition, since the analysis showed that the platform ultimate strength would likely be controlled by the foundation capacity, jacket joint strength was not assessed in the design level analysis.

A.5.2 Ultimate Strength Analysis

a) **Linear Analysis.** The ultimate strength linear analysis was performed in the same manner as the design level analysis; however, all safety factors were removed, the material yield strength was increased to 42 ksi, and the metocean loading was increased to the ultimate strength criteria. Results were obtained for six of the eight approach directions; two approach directions had such large deflections that convergence to a solution did not occur in a reasonable number of cycles. The output indicated that the piles had formed plastic hinges, whereas, none of the other members were overloaded.

b) **Nonlinear Analysis.**

1.0 Analysis Procedure

KARMA (INTRA) was used to perform a global inelastic static push-over analysis to assess the ultimate strength of the platform. This analysis was performed in accordance with the draft guidelines in Section 17.7.3. Modeling techniques used successfully in the past were adopted in this analysis. Specifically, the following idealizations were employed:

Deck elements. Linear beam elements (LBEM) were used to model the deck. The LBEM element is a linear elastic beam-column element capable of resisting axial, flexural, and twist forces. While the material response is linear, geometric nonlinearities were incorporated to account for P-delta effects. These effects were accounted for in all the elements considered in the analysis. Even though the deck was impacted by the environmental loads considered in this analysis, it was considered that the deck material response would likely be linear. Thus, the LBEM elements were employed to save computational time.

Braces in the jacket. Marshall B-strut elements(STRT) were used to model the braces in the jacket. The STRT element is a phenomenological post-buckling element which can be arbitrarily oriented in space. It is capable of resisting axial loads only. Sherman's equation was used to define the buckling response. The behavior of these braces show the greatest tendency toward solely axial response and these braces are likely to fail by axial buckling, if at all. Thus, STRT elements were employed.

Pile-jacket leg connections. Compatibility between the main piles and the jacket legs was provided by shear transfer (SHER) elements. These elements transfer shear loads only between the pile and the leg while allowing axial slippage.

Piles, legs, and all other members. Nonlinear beam elements (BEMC) were employed for the piles, legs, and all other members.

Foundation modeling. Self-aligning near field elements (SANE) were used to model the foundation. SANE elements can either model lateral and axial soil-pile interaction or pile tip bearing and suction response. The former is the P-Y/T-Z mode and the latter is the tip mode. The P-Y/T-Z mode consists of three orthogonal nonlinear elastic non-hysteretic springs. The tip mode consists of one such spring. The P-Y/T-Z springs are symmetric with positive and negative deformation while the tip springs are not.

Load cases. Since the platform is symmetric, the RSR is primarily determined by the magnitude of wave loading in a given direction. Therefore, only the four largest load cases from the linear ultimate strength results were analyzed. These four load cases are designated as load conditions 6, 11, 12, 13 in Figure 6. The analysis was performed by incrementally factoring the ultimate strength wave load (as defined by Draft Section 17) until the structural model recognized a loss of strength and stiffness past ultimate. As the load is increased, structural elements are checked and those experiencing inelastic behavior are recorded. This record can be examined to ensure proper modeling and to determine the failure mode of the structure.

2.0 Analytical Results

Reserve Strength Ratio (RSR) Based on API RP 2A-WSD, 20th Edition Loading

Figures 7 through 10 present the load-deflection curves and displaced shapes at failure for the four wave directions analyzed. The load-deflection curves are expressed in terms of the resultant base shear and the resultant deck displacement at leg A-1 elevation +43.00. Tables 6 through 9 provide additional information regarding the push-over analysis results for the various approach directions.

For all wave directions, the failure mode is characterized by nonlinear soil response followed by double-hinge pile failure. Since KARMA has no procedure for navigating a peak and convergence is difficult near the peak, the analyses were stopped at incipient failure. At this point, all of the main piles had failed.

The lowest RSR is 0.57 in the 270°(W) wave direction, which is the direction with the greatest base shear load. The RSR is also less than 1.0 for the 225°(SW) and 315°(NW) waves, while the 0°(N) wave produced an RSR of 1.21. Since the base shears for the 45° (NE), 90°(E), 135°(SE) and 180°(S) wave directions are lower than that of the 0°(N) wave, as shown in Figure 6, the RSRs will be greater than 1.0 for these directions.

Ultimate Strength According to API RP 2A Section 17 Loading Criteria

The maximum base shear on the platform calculated with the Section 17 Ultimate Strength metocean criteria (Table 1) was 3430 kips. Platform base shear capacities demonstrated by KARMA nonlinear push-over analyses are considerably less than this loading. Therefore, the KARMA analyses confirm the unacceptability of the members and foundation previously demonstrated by the Section 17 Design Level Analysis. It is apparent that the platform did not demonstrate adequate strength and stability to survive the ultimate strength loading criteria set forth in Sections 17.5 and 17.6. For that reason, the platform failed the assessment process.

A.6 Conclusions

Based on the Design Level Analysis and Ultimate Strength Analysis, the platform failed assessment. Further analytical refinements or mitigation actions must be taken to ensure adequacy for the current and extended use of this platform. Potential analytical refinements would include accounting for the foundation restraint provided by the conductors and using static criteria for the soil p-y curve generation. Mitigation actions are defined as modifications or operational procedures that reduce loads, increase capacities, or reduce consequences. Possible mitigation alternatives include platform strengthening, load reduction, and/or changes in exposure category. The platform will need to be reassessed after mitigation action is taken.

	API RP 2A-WSD Draft Section 17 Criteria for "Significant Environmental Impact/Manned-evacuated"		
Environmental Criteria	Design Level Analysis	Ultimate Strength Analysis	API RP 2A-WSD 20th Edition Criteria
Classification	Full Population Hurricanes	Full Population Hurricanes	Full Population Hurricanes
Wave Height (ft.)	44.1 to 54	46.9 to 67	44.1 to 63
Wave Period (sec.)	12.1	13.5	13.0
Current (kts)	1.6 (0.2 minimum)	2.3 (0.2 minimum)	2.1 (0.2 minimum)
Wave & Current Direction	Omni-Directional	Directional (20th Ed.)	Directional
Wave Kinematics Factor	0.88	0.88	0.88
Drag Coef., Cd (smooth)	0.65	0.65	0.65
Drag Coef., Cd (rough)	1.05	1.05	1.05
Inertia Coef., Cm (smooth)	1.6	1.6	1.6
Inertia Coef., Cm (rough)	1.2	1.2	1.2
Current Blockage Factor	0.8 to 0.85	0.8 to 0.85	0.8 to 0.85
Conductor Shielding Factor	0.9	0.9	0.9
Marine Growth Thickness (in.)	1.5	1.5	1.5
Marine Growth Roughness (in.)	0.6	0.6	0.6
Storm Tide (ft.)	2.5	2.5	3.6
Wind Velocity, 1-hr (kts)	65	85	80
Minimum Deck Height (ft.)	N/A	45.8	51

Table 1. Metocean Loading Criteria, Gulf of Mexico, Water Depth = 157 ft.

Wave Approach Direction (deg.)	0	45	90	135	180	225	270	315
Orientation w.r.t. Platform North (towards)	NW	N	NE	E	SE	S	SW	W
Wave Height, Hmax (ft)	57.0	46.9	46.9	46.9	50.3	60.3	67.0	63.7
Wave Period, T (sec.)	13.5	13.5	13.5	13.5	13.5	13.5	13.5	13.5
Current Velocity (ft/sec)	0.34	0.34	0.34	0.34	0.34	3.88	3.88	3.88
Current Direction (deg.)	0	45	90	135	180	270	270	270
Storm surge (ft)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Wind Velocity @ 10 m (ft/sec)	143.5	143.5	143.5	143.5	143.5	143.5	143.5	143.5
Wave Load in Deck (kips)	0	0	0	0	0	0	98.6	0

Table 2. Environmental Loading Data for Ultimate Strength Analysis
(Based on Draft Section 17)

Description	Weight (Kips)
Deck Deadload	350
Jacket Deadload	941
Jacket Buoyancy	-491
Deck Liveload	1425
Piles Below Mudline	598
Total	2823

Table 3: Gravity Loading Summary

Angle (degrees)	Angle (radians)	V (ft./sec.)	Crest Height (ft.)	H (ft.)	Ax (ft.^2)	Ay (ft.^2)	A (ft.^2)	Alpha cpf	Cd	Fdk (kips)	Zdk (ft.)
0	0.00	16.60	27.67	187.17	0.00	0.00	0.00	0.85	2.00	NO FORCE	NO FORCE
45	0.79	16.60	27.67	187.17	0.00	0.00	0.00	0.80	1.50	NO FORCE	NO FORCE
90	1.57	16.60	27.67	187.17	0.00	0.00	0.00	0.85	2.00	NO FORCE	NO FORCE
135	2.36	21.50	34.84	194.32	0.00	0.00	0.00	0.80	1.50	NO FORCE	NO FORCE
180	3.14	20.70	32.53	192.03	0.00	0.00	0.00	0.85	2.00	NO FORCE	NO FORCE
225	3.93	26.40	42.77	202.27	60.50	74.92	95.75	0.80	1.50	98.62	6.14
270	4.71	22.90	37.49	196.99	0.00	0.00	0.00	0.85	2.00	NO FORCE	NO FORCE
315	5.50	18.20	30.01	189.51	0.00	0.00	0.00	0.80	1.50	NO FORCE	NO FORCE

Where: V=maximum wave-induced horizontal fluid velocity at the crest elevation or the top of the main deck silhouette, which ever is lower

H=elevation of the crest height

Ax=X projected area*

Ay=Y projected area*

A=Axcos (angle) + Aysin (angle)*

Fdk=(1/2*1.98*Cd(0.88*V+alphacbf*U)^2)*A

Zdk=50% of the distance between the lowest point of the silhouette area and the lower of the wave crest or top of the main deck.

Table 4: Wave/Current Deck Force Calculations

Description	Design Level Analysis	Ultimate Strength Analysis
Material yield strength	36 ksi	42 ksi
Member capacity estimate: *Braces * Legs/Piles * Piles	as per Section 3 as per Section 3 as per Section 6.4	
Joint capacity estimate:~	as per Section 4	
Soil spring (p-y, t-z, q-z) generation	as per Section 6.8.3	

~Jacket joint strength was not assessed.

Table 5: Member Capacity Estimation Data

Analysis Case: 3-D Model
Load Condition: 1 - N Approach
Lateral load level at failure of the first member (681-1181) = 1180 kips

Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips
1	0.0011	0.0038	17	1.9494	1343	33	3.0222	1540	49	3.4364	1581
2	0.0023	0.0076	18	1.9962	1360	34	3.0762	1548	50	3.4570	1583
3	0.0035	0.0114	19	2.0445	1376	35	3.1550	1556	51	3.4776	1584
4	0.0046	0.0153	20	2.0941	1392	36	3.1735	1558	52	3.5004	1585
5	0.0058	0.0193	21	2.1616	1409	37	3.1863	1560	53	3.5224	1587
6	0.1261	131	22	2.2370	1426	38	3.2183	1563	54	3.5444	1590
7	0.2656	262	23	2.3204	1442	39	3.2361	1564	55	3.5558	1590
8	0.4175	393	24	2.4166	1458	40	3.2559	1566	56	3.5650	1591
9	0.5791	524	25	2.5348	1474	41	3.2723	1568	57	3.5764	1591
10	0.7507	655	26	2.5974	1483	42	3.2907	1570	58	3.5878	1592
11	0.9311	786	27	2.6758	1491	43	3.3113	1571	59	3.5999	1593
12	1.1196	917	28	2.7441	1498	44	3.3319	1573	60	3.6292	1594
13	1.3278	1048	29	2.8039	1507	45	3.3526	1574	61	3.6649	1595
14	1.5652	1179	30	2.8630	1515	46	3.3739	1576	62	3.6985	1595
15	1.8636	1311	31	2.9170	1524	47	3.3952	1578	63	3.7343	1597
16	1.9054	1327	32	2.9703	1532	48	3.4158	1580	64	3.7694	1597

Table 6: Ultimate Strength Analysis - 0 Degree Approach Direction

Analysis Case: 3-D Model
Load Condition: 2 - SW Approach
Lateral load level at failure of the first member(621-1121)= 1317 kips

Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips
1	0.0011	0.0038	14	1.8060	1317	27	3.2912	1584	40	3.4723	1593
2	0.0023	0.0076	15	1.8720	1344	28	3.3052	1585	41	3.4793	1594
3	0.0035	0.0114	16	1.9440	1370	29	3.3182	1586	42	3.4853	1594
4	0.0046	0.0153	17	2.0340	1396	30	3.3402	1587	43	3.4923	1594
5	0.0058	0.0193	18	2.1480	1423	31	3.3702	1589	44	3.5063	1595
6	0.1278	132	19	2.3370	1449	32	3.3962	1590	45	3.5253	1595
7	0.2630	264	20	2.5691	1476	33	3.4242	1591	46	3.5443	1596
8	0.4099	395	21	2.7561	1502	34	3.4342	1592	47	3.5633	1597
9	0.5671	527	22	2.9251	1528	35	3.4413	1592	48	3.5823	1597
10	0.7338	659	23	3.0891	1555	36	3.4473	1592	49	3.6053	1598
11	0.9089	790	24	3.2642	1581	37	3.4543	1592	50	3.6283	1599
12	1.0920	922	25	3.2772	1582	38	3.4603	1593	51	3.6514	1599
13	1.2910	1054	26	3.2772	1582	39	3.4663	1593	52	3.6734	1600
									53	3.6964	1601

Table7: Ultimate Strength Analysis - 225 Degree Approach Direction

Analysis Case: 3-D Model
Load Condition: 3 - W Approach
Lateral load level at failure of the first member (621-1121) = 974 kips

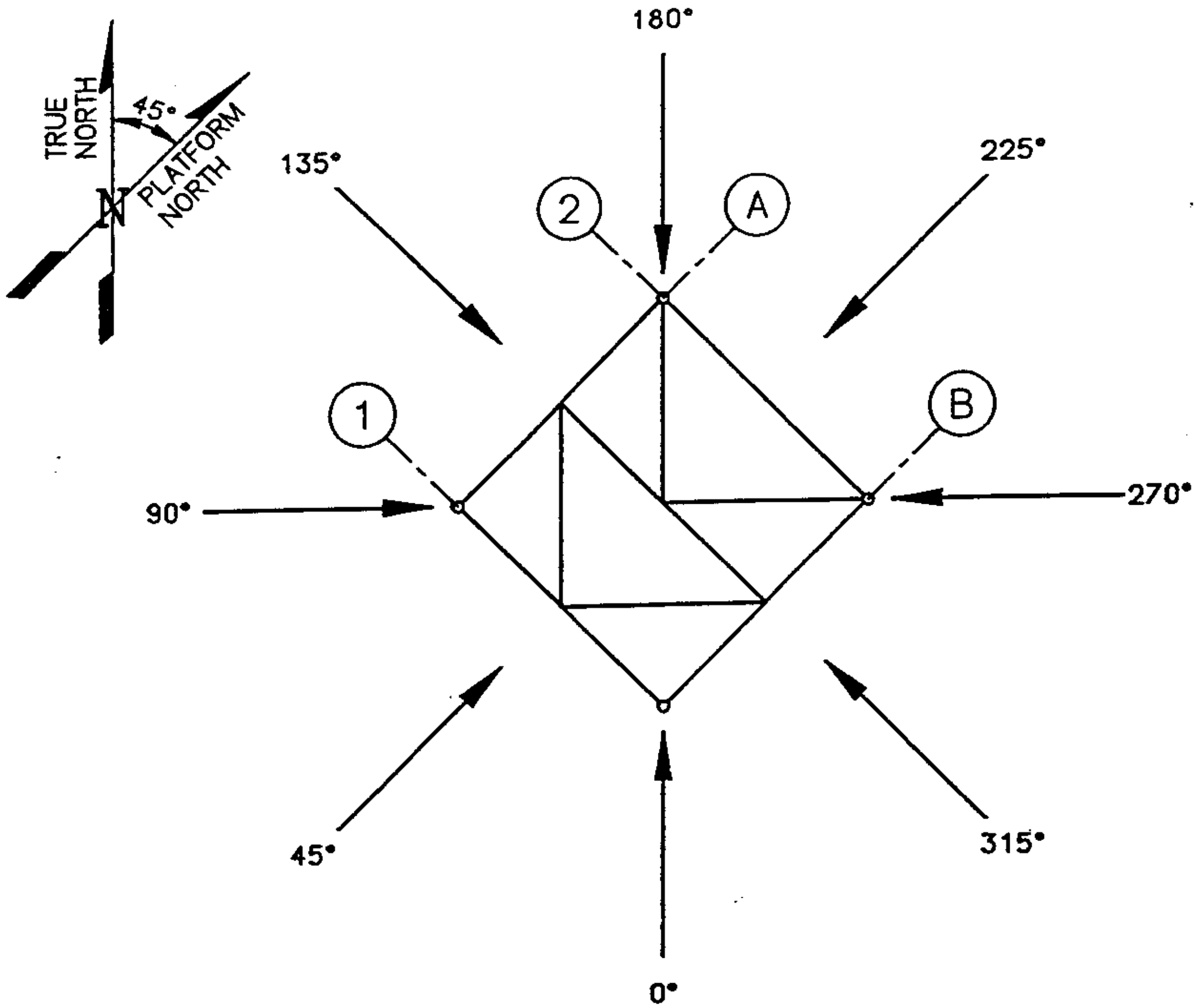
Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips
1	0.0011	0.0038	21	1.5430	1191	41	2.4213	1484	61	2.8902	1555
2	0.0023	0.0076	22	1.5797	1209	42	2.4531	1487	62	2.9256	1559
3	0.0035	0.0114	23	1.6151	1227	43	2.4849	1491	63	2.9575	1563
4	0.0046	0.0153	24	1.6511	1245	44	2.5132	1494	64	2.9879	1566
5	0.0058	0.0193	25	1.6900	1263	45	2.5401	1498	65	3.0204	1570
6	0.1024	108	26	1.7282	1281	46	2.5599	1501	66	3.0530	1573
7	0.2130	217	27	1.7721	1299	47	2.5903	1505	67	3.0891	1577
8	0.3317	325	28	1.8152	1317	48	2.6101	1508	68	3.1273	1580
9	0.4576	433	29	1.8583	1335	49	2.6384	1513	69	3.1648	1584
10	0.5902	541	30	1.9043	1353	50	2.6561	1515	70	3.2023	1588
11	0.7287	650	31	1.9531	1371	51	2.6837	1520	71	3.2214	1590
12	0.8730	758	32	2.0019	1389	52	2.7014	1522	72	3.2426	1592
13	1.0215	866	33	2.0578	1407	53	2.7219	1527	73	3.2631	1593
14	1.1804	974	34	2.1306	1426	54	2.7466	1530	74	3.2830	1595
15	1.3513	1083	35	2.2049	1444	55	2.7643	1534	75	3.3042	1597
16	1.3819	1101	36	2.2395	1451	56	2.7884	1538	76	3.3240	1599
17	1.4124	1119	37	2.2749	1458	57	2.8054	1541	77	3.3445	1601
18	1.4447	1137	38	2.3110	1465	58	2.8294	1545	78	3.3643	1602
19	1.4758	1155	39	2.3541	1472	59	2.8478	1548	79	3.3842	1604
20	1.5104	1173	40	2.4015	1479	60	2.8719	1552	80	3.4181	1606

Table 8: Ultimate Strength Analysis - 270 Degree Approach Direction

Analysis Case: 3-D Model
Load Condition: 4 -NW Approach
Lateral load level at failure of the first member (681-1181) = 1060 kips

Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips	Load Step	Lateral Displacement at Deck Level (+43') at South East Leg ft.	Lateral Load Kips
1	0.0011	0.0038	18	1.0633	926	35	1.5839	1248	52	2.7074	1518
2	0.0023	0.0076	19	1.0903	945	36	1.6240	1267	53	2.7165	1520
3	0.0035	0.0114	20	1.1164	964	37	1.6651	1286	54	2.7265	1522
4	0.0046	0.0153	21	1.1444	983	38	1.7072	1304	55	2.7375	1524
5	0.0058	0.0193	22	1.1714	1002	39	1.7523	1323	56	2.7476	1526
6	0.1752	189	23	1.1994	1021	40	1.8004	1342	57	2.7576	1527
7	0.3762	378	24	1.2274	1040	41	1.8566	1361	58	2.7677	1529
8	0.5971	567	25	1.2564	1059	42	1.9159	1380	59	2.7777	1531
9	0.8356	756	26	1.2855	1078	43	1.9893	1399	60	2.7907	1533
10	0.8602	775	27	1.3165	1096	44	2.0818	1418	61	2.8068	1535
11	0.8851	794	28	1.3455	1115	45	2.2014	1437	62	2.8290	1537
12	0.9099	813	29	1.3766	1134	46	2.3311	1456	63	2.8481	1539
13	0.9350	832	30	1.4067	1153	47	2.4486	1475	64	2.8653	1541
14	0.9601	851	31	1.4397	1172	48	2.5661	1493	65	2.8834	1543
15	0.9856	870	32	1.4748	1191	49	2.6745	1512	66	2.9016	1545
16	1.0113	889	33	1.5088	1210	50	2.6854	1514	67	2.9208	1546
17	1.0363	907	34	1.5449	1229	51	2.6964	1516	68	2.9390	1548
									69	2.9572	1550

Table 9: Ultimate Strength Analysis - 315 Degree Approach Direction

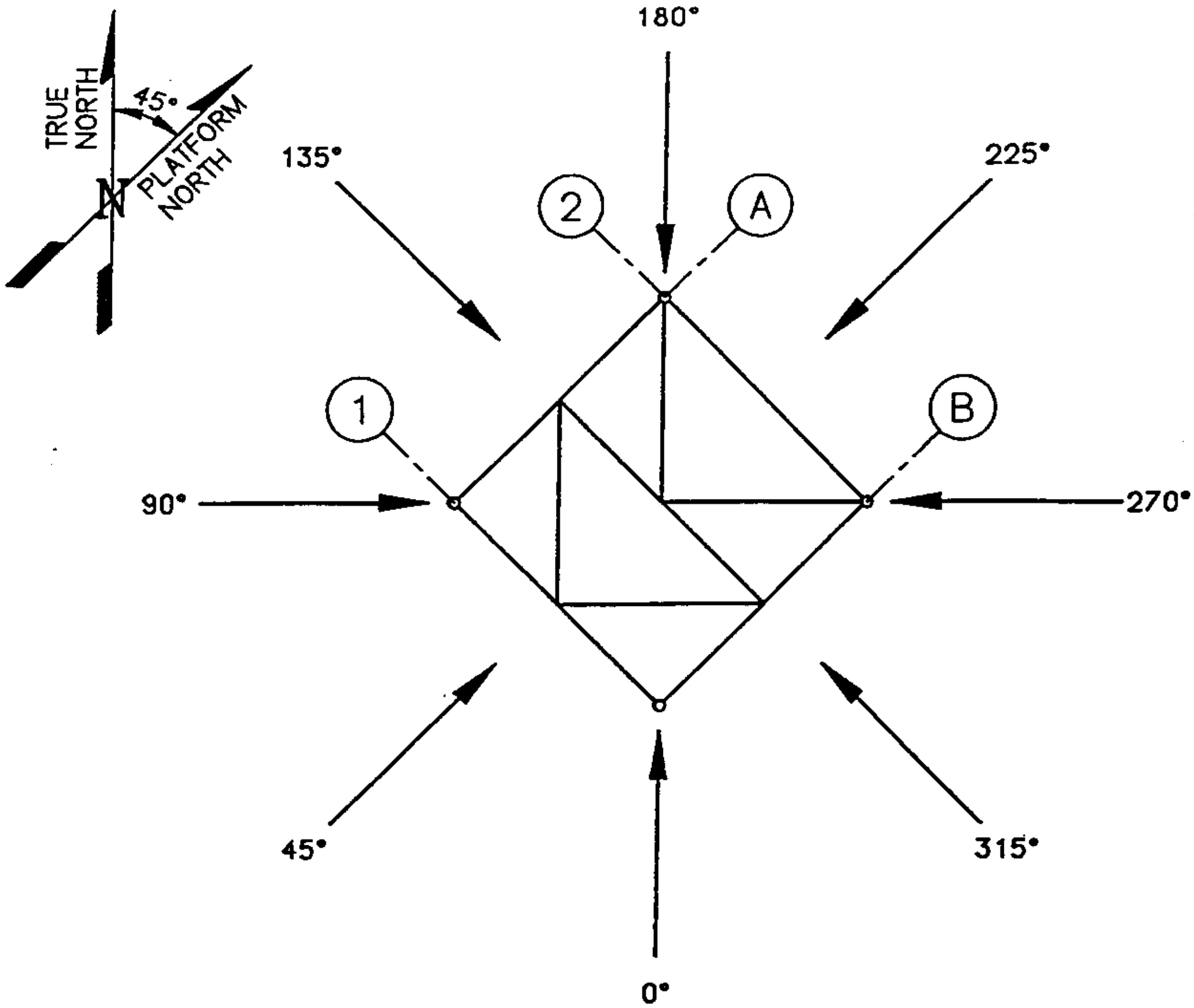


Wave Approach Direction (deg.)	0	45	90	135	180	225	270	315
Wave Height, Hmax (ft)	53.6	44.1	44.1	44.1	47.3	54.0	54.0	54.0
Current Velocity (kts)	0.2	0.2	0.2	0.2	0.2	1.6	1.6	1.6
Current Direction (deg.)	0	45	90	135	180	225	270	315

Wave Height and Current for Design Level Analysis

(Based on Draft Section 17; refer to Table 1 for other metocean parameters)

Figure 1

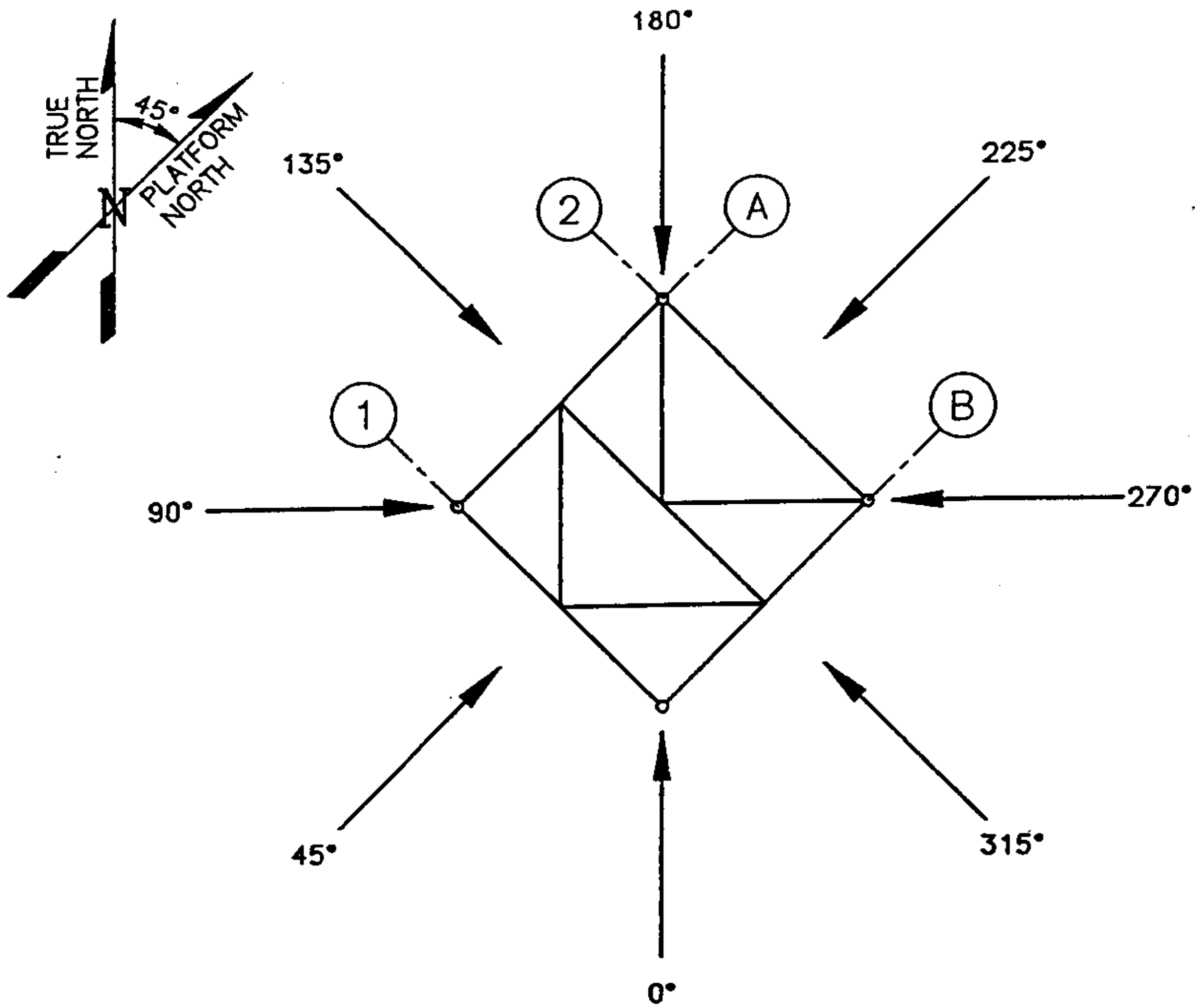


Wave Approach Direction (deg.)	0	45	90	135	180	225	270	315
Wave Height, Hmax (ft)	57.0	46.9	46.9	46.9	50.3	60.3	67.0	63.7
Current Velocity (kts)	0.2	0.2	0.2	0.2	0.2	2.3	2.3	2.3
Current Direction (deg.)	0	45	90	135	180	270	270	270

Wave Height and Current for Ultimate Strength Analysis

(Based on Draft Section 17; refer to Table 1 for other metocean parameters)

Figure 2



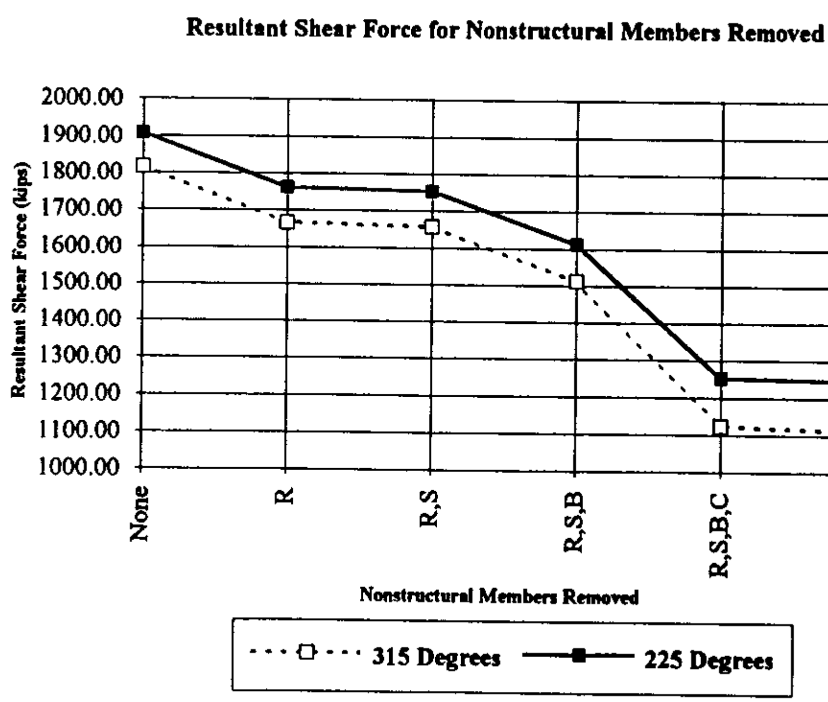
Wave Approach Direction (deg.)	0	45	90	135	180	225	270	315
Wave Height, Hmax (ft)	53.6	44.1	44.1	44.1	47.3	56.7	63.0	59.9
Current Velocity (kts)	0.2	0.2	0.2	0.2	0.2	2.1	2.1	2.1
Current Direction (deg.)	0	45	90	135	180	270	270	270

Wave Height and Current from API RP 2A, 20th Ed.

(Refer to Table 1 for other metocean parameters)

Figure 3

Comments	Items Removed	Abbrev. for Graphs	315 Degree Approach case			225 Degree Approach case		
			Resultant Shear Force (kips)	Change in Shear Force (kips)	Resultant Overturning Moment (ft.-kips)	Resultant Shear Force (kips)	Change in Shear Force (kips)	Resultant Overturning Moment (ft.-kips)
Entire Structure	None	None	1816.00		226510.56	1906.84		234785.00
	Risers	R	1666.17	149.83	209056.59	1761.07	145.77	217894.81
	Risers, Stairs	R,S	1656.63	9.54	207332.49	1753.53	7.54	216563.00
	Risers, Stairs, Boat Landing	R,S,B	1511.32	145.31	184146.43	1611.78	141.75	193929.69
	Risers, Stairs, Boat Landing, Conductors	R,S,B,C	1123.51	387.81	136183.49	1251.72	360.06	149794.56
No NONSTR	Risers, Stairs, Boat Landing, Conductors,& all other Nonstr	All	1110.33	13.18	134006.31	1245.34	6.38	148740.62



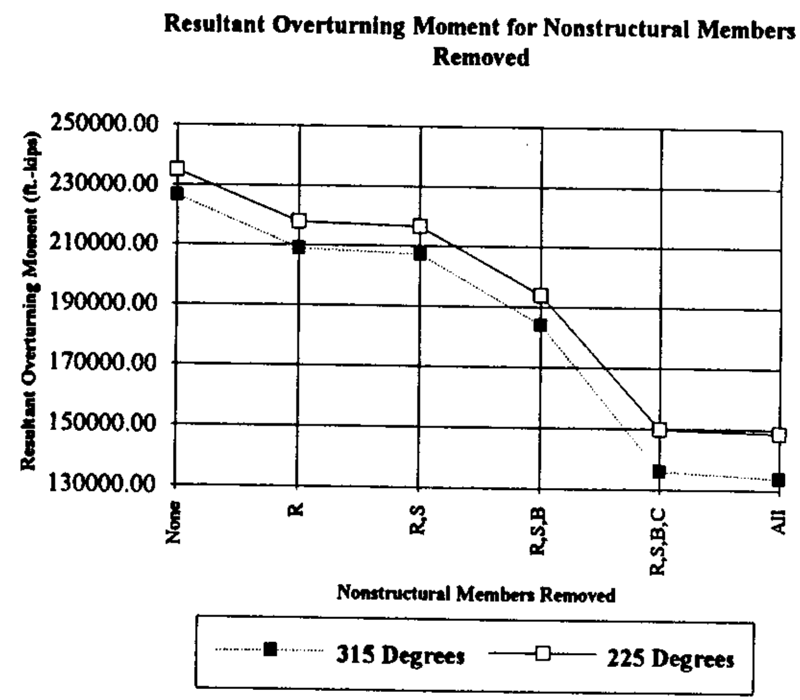
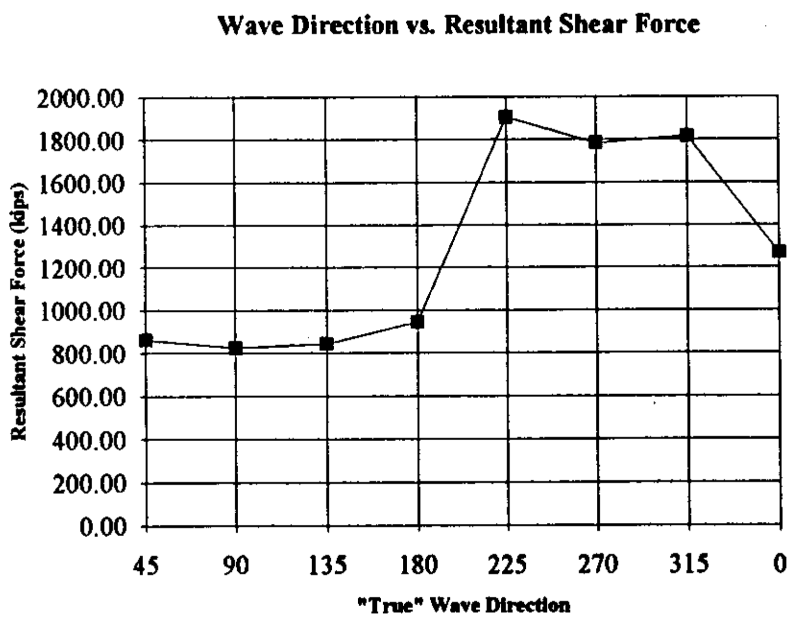


Figure 4: Forces on Nonstructural Groups

Load Condition	Maximum Shear: Trial Number	Maximum Shear: Crest Position	Maximum Moment: Trial Number	Maximum Moment: Crest Position	Platform Angle* Wave Direction	"True" North Wave Direction	Maximum Base Shear: X (kips)	Maximum Base Shear: Y (kips)	Maximum Base Shear: Resultant (kips)	Maximum Overturning Moment: X (ft-kips)	Maximum Overturning Moment: Y (ft-kips)	Maximum Overturning Moment: Resultant (ft-kips)
6	8	-20.00	8	-20.00	135	0	-888.71	903.69	1267.46	-118340.31	-117400.00	166694.88
7	7	-15.00	7	-15.00	90	45	-0.95	860.79	860.79	-109077.06	-104.89	109072.06
8	6	-20.00	7	-15.00	45	90	578.71	586.71	824.26	-74635.38	74585.94	105515.38
9	6	-20.00	7	-15.00	0	135	843.03	-0.98	843.03	194.77	108457.88	108457.69
10	6	-20.00	7	-15.00	315	180	663.97	-674.68	946.60	86303.25	85822.87	121712.00
11	7	-15.00	9	-5.00	270	225	0.41	-1906.86	1906.84	234785.00	61.73	234785.00
12	7	-15.00	8	-10.00	225	270	-1247.50	-1273.34	1782.60	157651.00	-155762.12	221620.56
13	7	-15.00	8	-10.00	180	315	-1816.00	0.83	1816.06	-46.50	-226510.56	226510.56

* Platform Angle = wave counterclockwise from -x direction (towards)



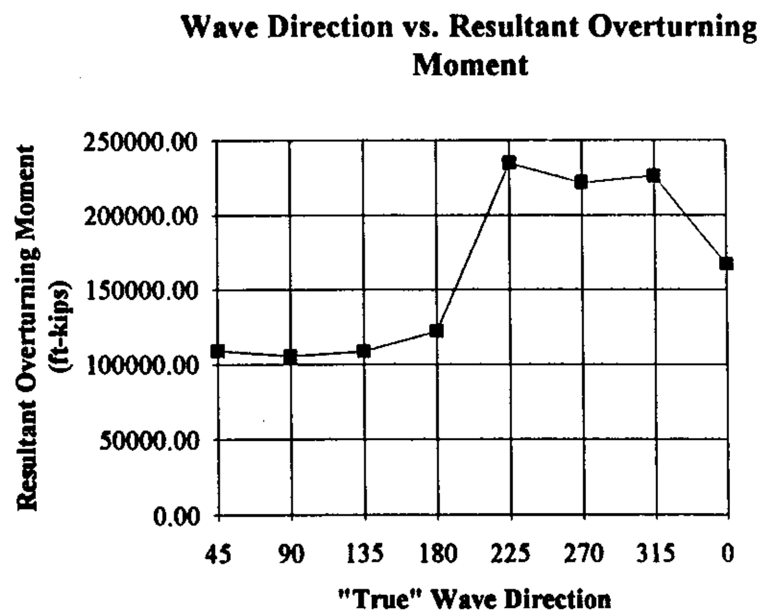


Figure 5: Design Level - Base Shear & Overturning Moment vs. Approach Direction

Load Condition	Maximum Shear Trial Number	Maximum Shear Crest Position	Maximum Moment Trial Number	Maximum Moment Crest Position	Platform Angle* Wave Direction	"True" North Wave Direction	Maximum Shear X (kips)	Maximum Shear Y (kips)	Maximum Shear Resultant (kips)	Wave Deck Force X (kips)	Wave Deck Force Y (kips)	Wave Deck Force Result. a (kips)	Result. Base Shear b (kips)	Zdk** (ft.)	Maximum Moment X (ft-kips)	Maximum Moment Y (ft-kips)	Maximum Moment Resultant (ft-kips)	Wave Deck Moment X c (ft-kips)	Wave Deck Moment Y d (ft-kips)	Wave Deck Moment Result. e (ft-kips)	Resultant Overturning Moment f (ft-kips)
6	7	-15.00	8	-10.00	135	0	-1102.64	1117.62	1569.99	0.00	0.00	0.00	1569.99	0.00	-145178.37	-144891.00	205110.12	0.00	0.00	0.00	205110.12
7	6	-20.00	7	-15.00	90	45	-0.96	1057.04	1057.04	0.00	0.00	0.00	1057.04	0.00	-131973.69	-108.21	131973.69	0.00	0.00	0.00	131973.73
8	6	-20.00	7	-15.00	45	90	712.74	720.76	1013.66	0.00	0.00	0.00	1013.66	0.00	-90438.44	90878.00	128210.44	0.00	0.00	0.00	128210.46
9	6	-20.00	7	-15.00	0	135	1037.19	-1.04	1037.19	0.00	0.00	0.00	1037.19	0.00	205.19	131917.06	131917.19	0.00	0.00	0.00	131917.22
10	6	-20.00	7	-15.00	315	180	815.66	-827.07	1161.62	0.00	0.00	0.00	1161.62	0.00	104130.44	104096.25	147238.50	0.00	0.00	0.00	147238.51
11	7	-15.00	8	-10.00	270	225	-353.86	-2579.81	2603.97	0.00	0.00	0.00	2603.97	0.00	320025.44	-38606.87	322345.69	0.00	0.00	0.00	322345.73
12	7	-15.00	7	-15.00	225	270	-2335.66	-2373.98	3330.33	-69.73	-69.73	98.62	3428.94	6.14	299055.56	-297358.50	421730.06	13676.84	-13676.84	19343.33	441072.03
13	7	-15.00	8	-10.00	180	315	-1885.63	-283.42	1906.80	0.00	0.00	0.00	1906.80	0.00	30401.58	-238367.06	240297.94	0.00	0.00	0.00	240297.96

* Platform Angle = wave counterclockwise from -x direction (towards)

** Zdk = distance above elevation +33.00 that the wave deck force is applied

a - Wave Deck Resultant Force taken from WAVEFRC.XLS

b - Resultant Base Shear is calculated as follows:

((Max. shear X + Wave deck force X)^2 + (Max. shear Y + Wave deck force Y)^2)^.5

c - Wave Deck Moment X is calculated as follows:

-(Wave Deck Force Y)*(Zdk+190)

d - Wave Deck Moment Y is calculated as follows:

(Wave Deck Force X)*(Zdk+190)

e - Wave Deck Resultant Moment is calculated as follows:

((Wave deck moment X)^2 + (Wave deck moment Y)^2)^.5

f - Resultant Overturning Moment is calculated as follows:

((Max. moment X + Wave deck moment X)^2 + (Max. moment Y + Wave deck moment Y)^2)^.5

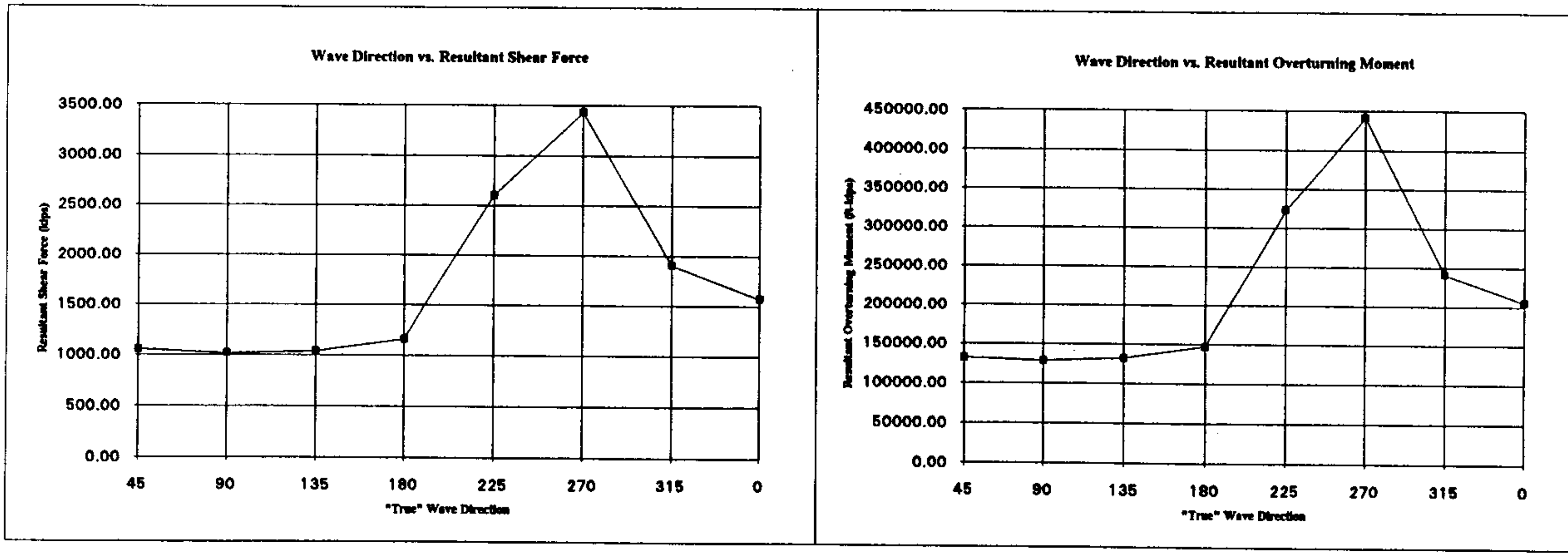
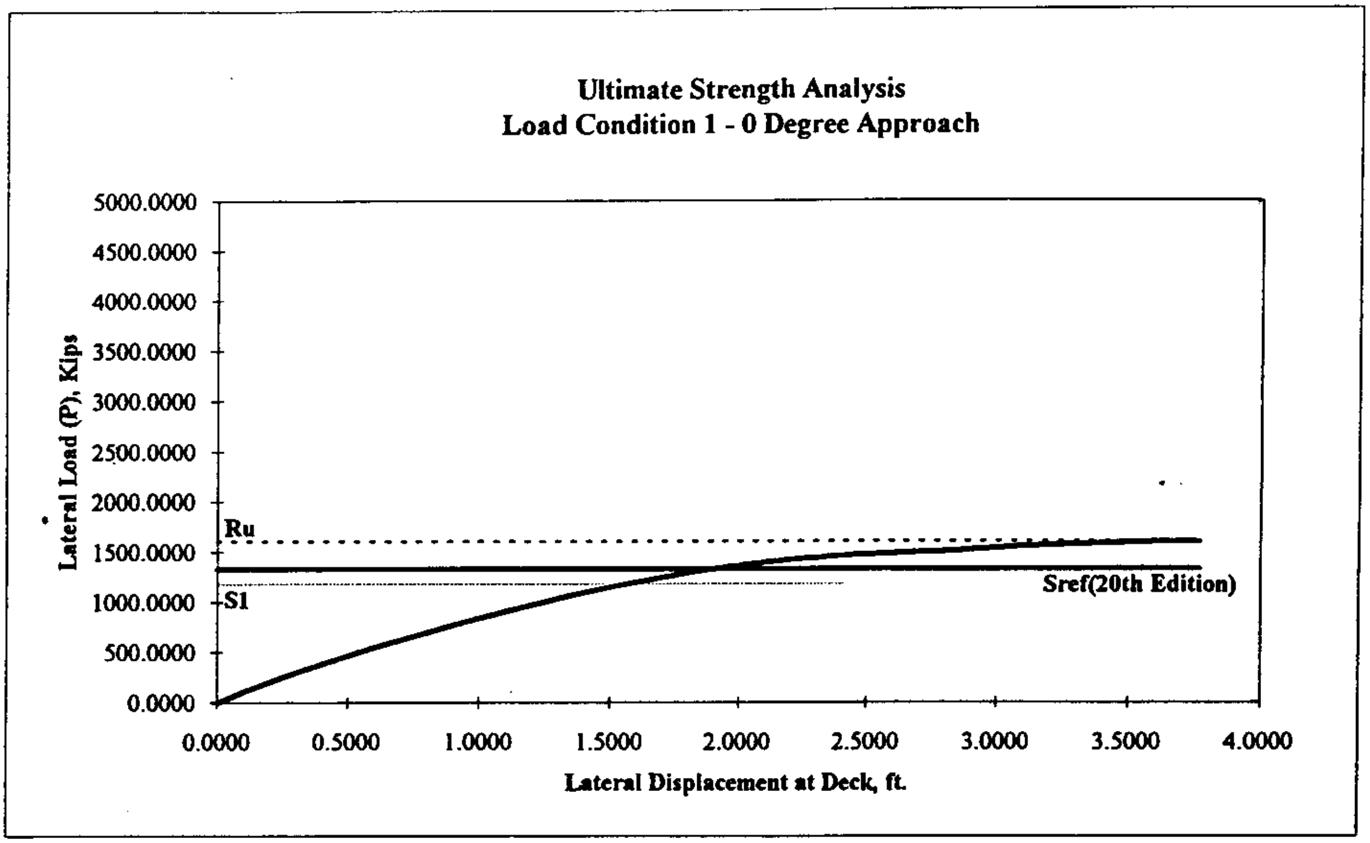
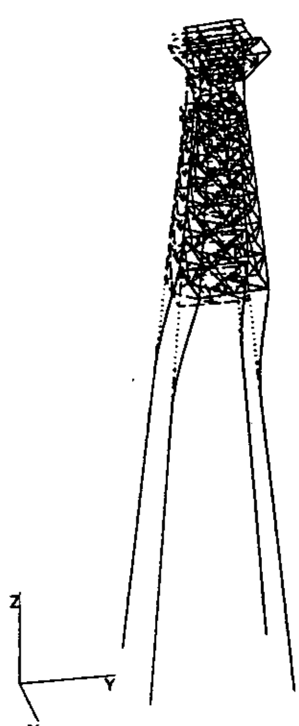


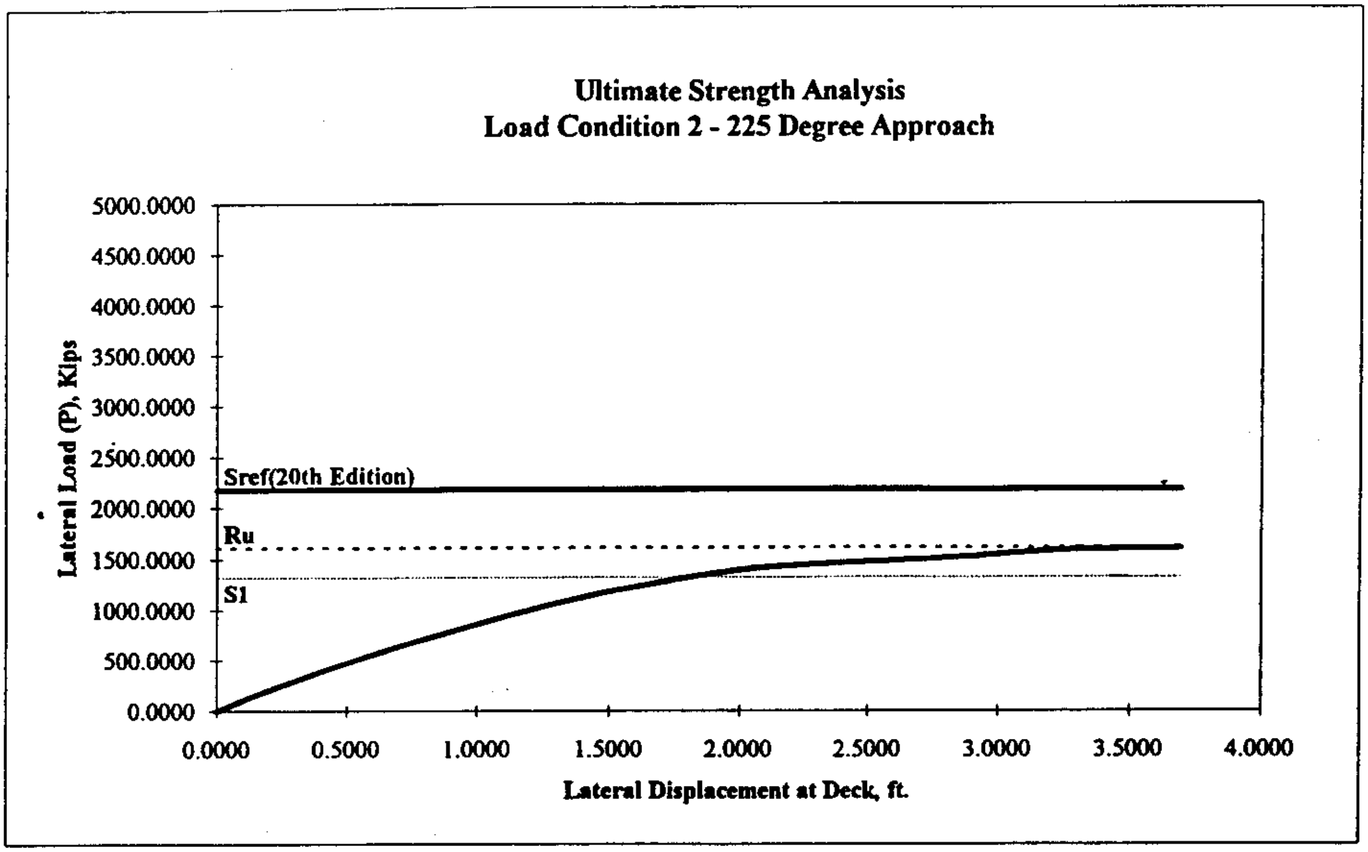
Figure 6: Ultimate Strength - Base Shear & Overturning Moment vs. Approach Direction

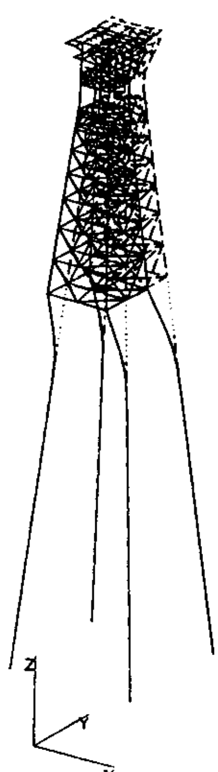




Reference Level Load (Sref)	1325 kips
Load Level at First Member Failure	1180 kips
Ultimate Capacity (Ru)	1600 kips
Reserve Strength Ratio (RSR)	1.21
Platform Failure Mode:	Pile,Soil

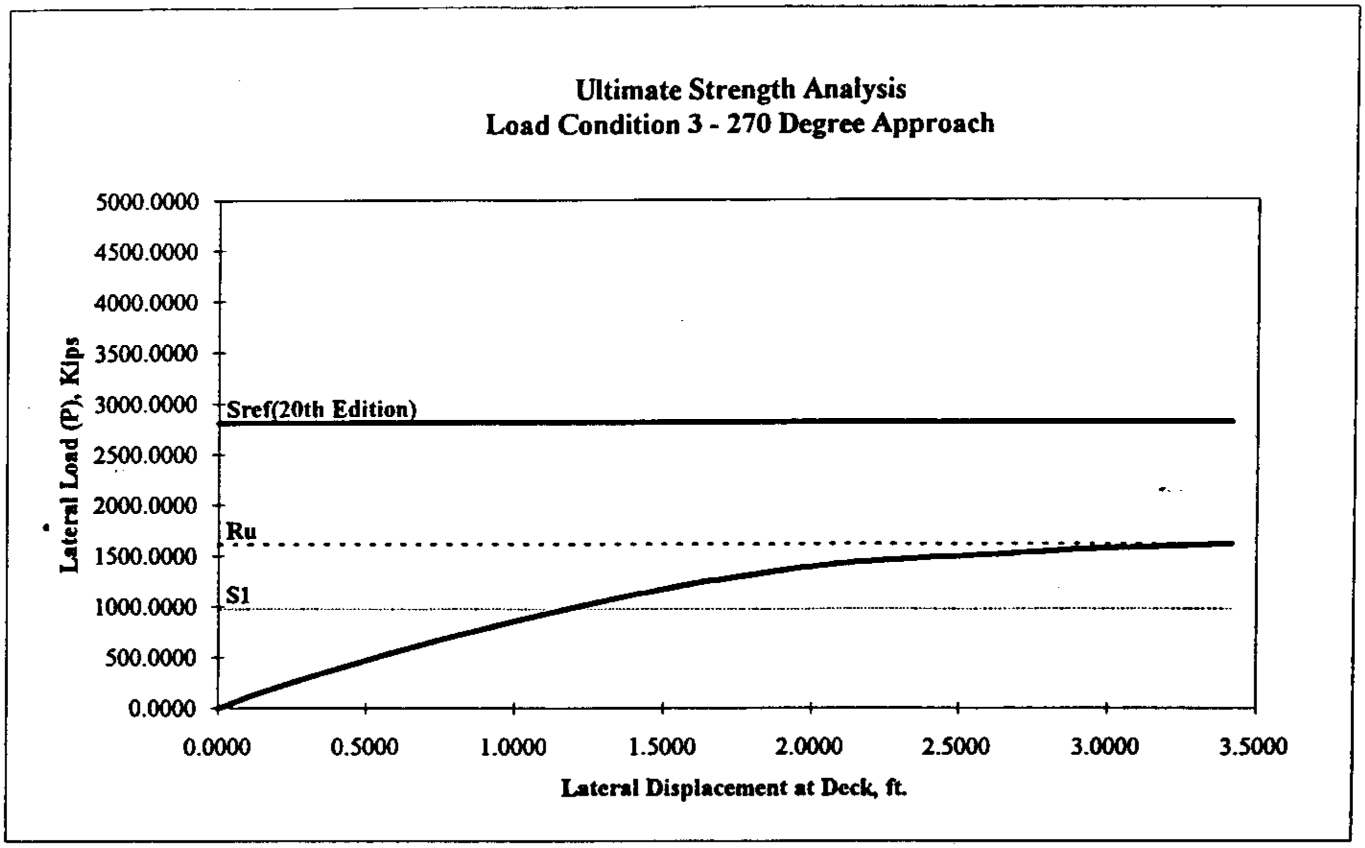
Figure 7 : Load Condition 1 - 0 Degree Approach

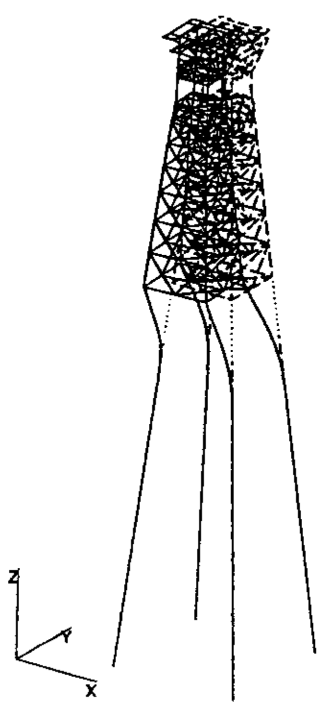




Reference Level Load (Sref)	2174 kips
Load Level at First Member Failure	1317 kips
Ultimate Capacity (Ru)	1610 kips
Reserve Strength Ratio (RSR)	0.74
Platform Failure Mode:	Pile,Soil

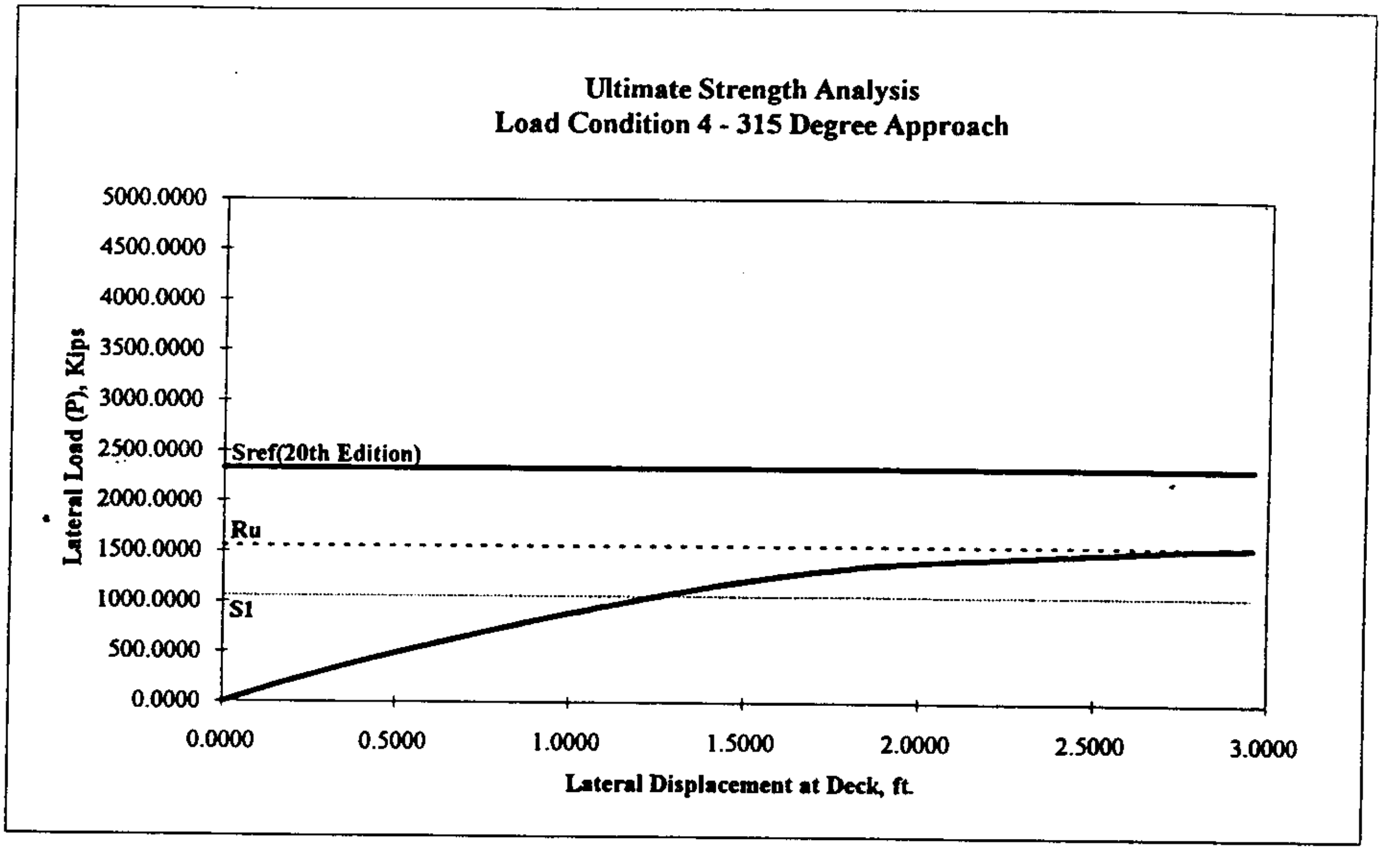
Figure 8: Load Condition 2 - 225 Degree Approach

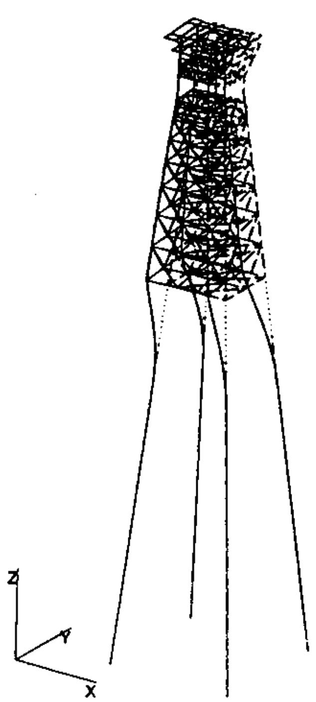




Reference Level Load (Sref)	2810 kips
Load Level at First Member Failure	980 kips
Ultimate Capacity (Ru)	1610 kips
Reserve Strength Ratio (RSR)	0.57
Platform Failure Mode:	Pile,Soil

Figure 9: Load Condition 3 - 270 Degree Approach





Reference Level Load (Sref)	2325 kips
Load Level at First Member Failure	1060 kips
Ultimate Capacity (Ru)	1550 kips
Reserve Strength Ratio (RSR)	0.67
Platform Failure Mode:	Pile,Soil

Figure 10: Load Condition 4 - 315 Degree Approach

Appendix B

Design Level -

Maximum Combined Unity Checks

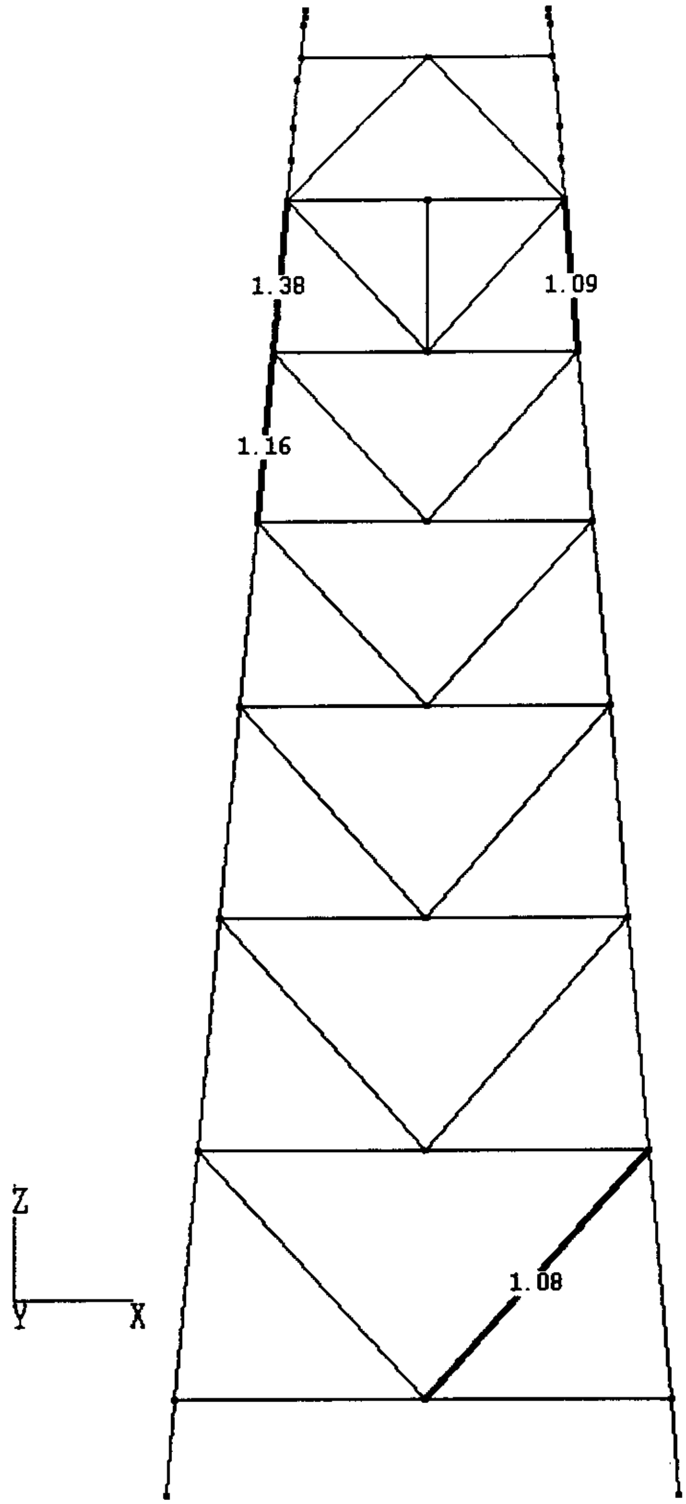


Figure B-1: Benchmark Platform - Column Row 1 - Jacket"

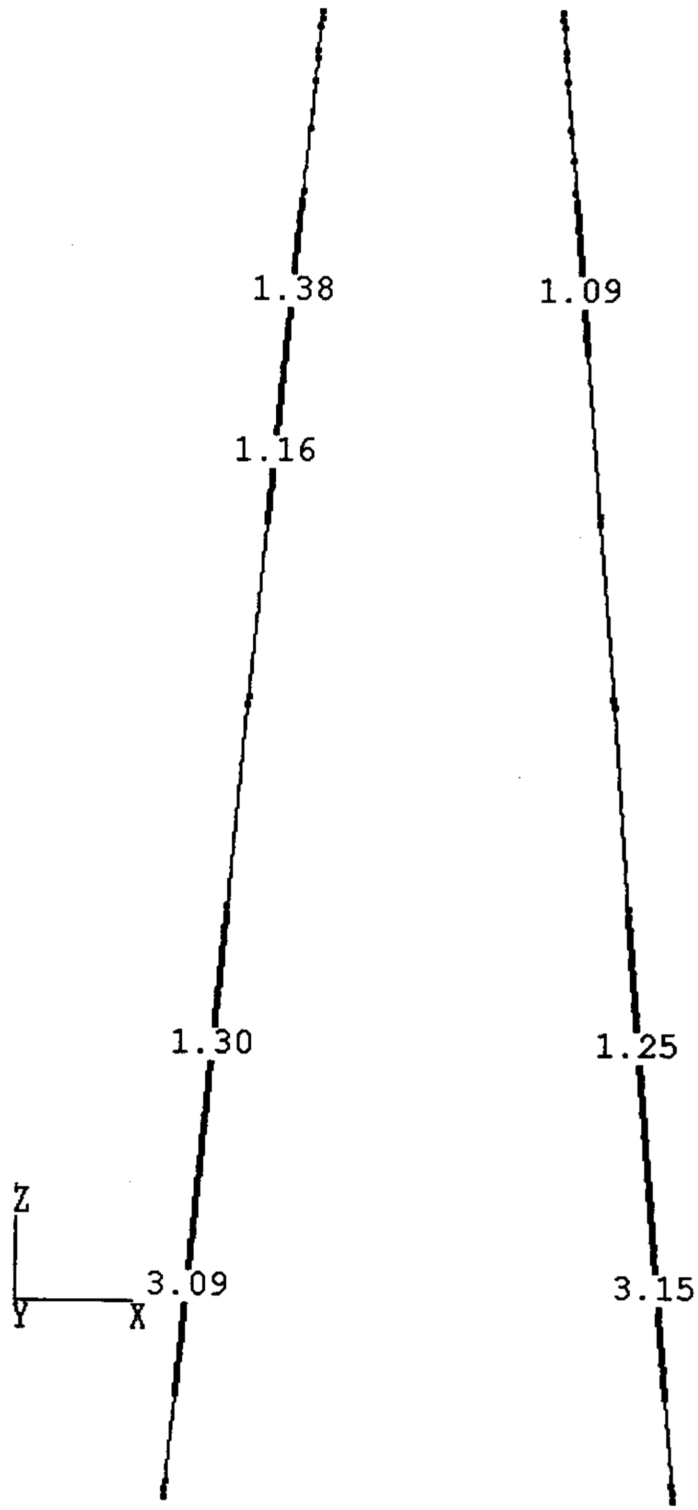


Figure B-2: Benchmark Platform - Column Row 1 - Pile"

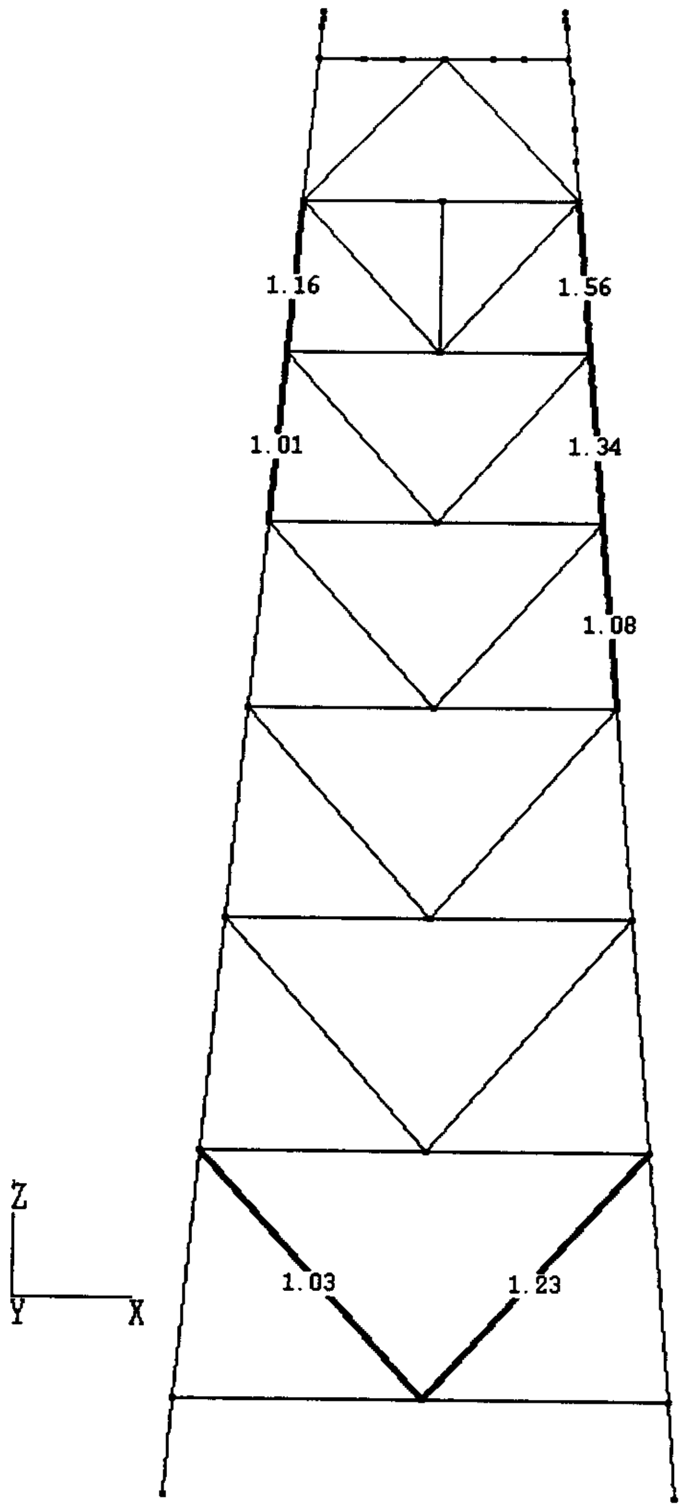


Figure B-3: Benchmark Platform - Column Row 2 - Jacket

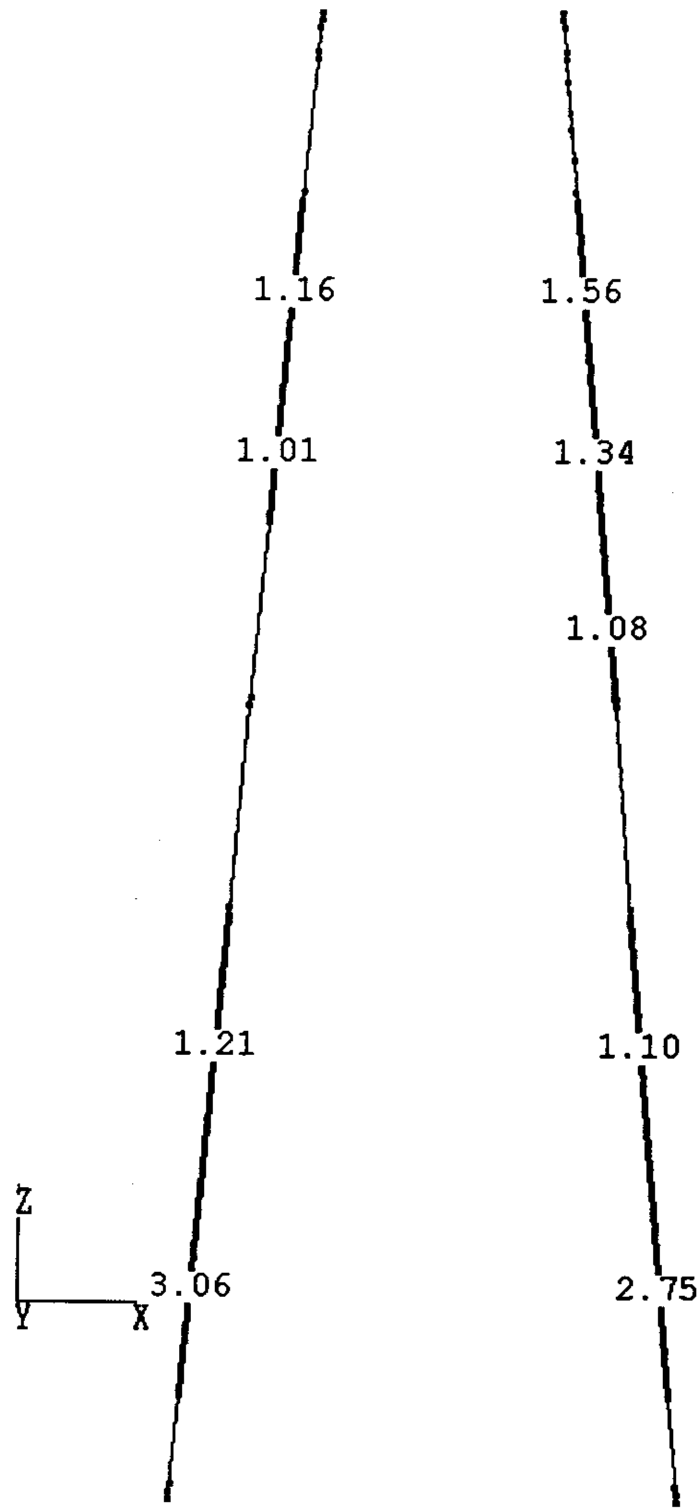


Figure B-4: Benchmark Platform - Column Row 2 - Pile

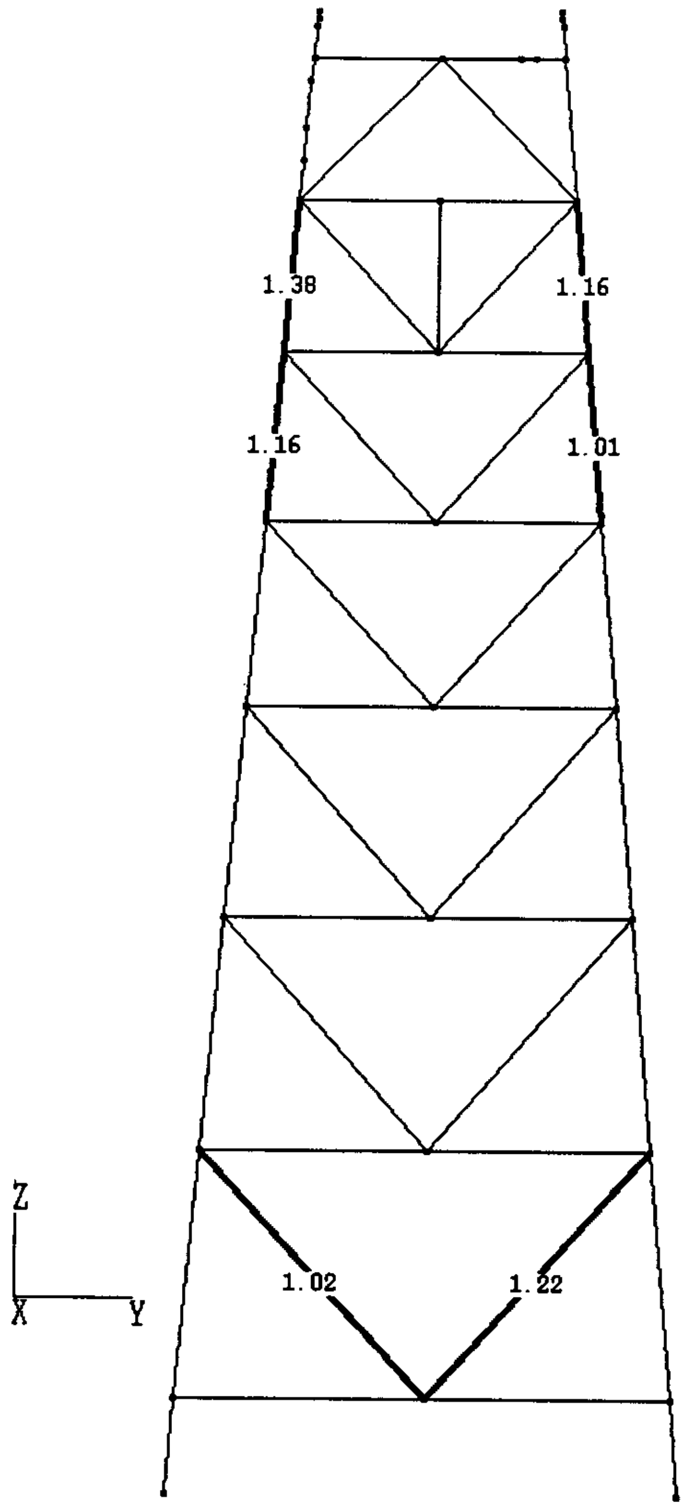


Figure B-5: Benchmark Platform - Column Row A - Jacket

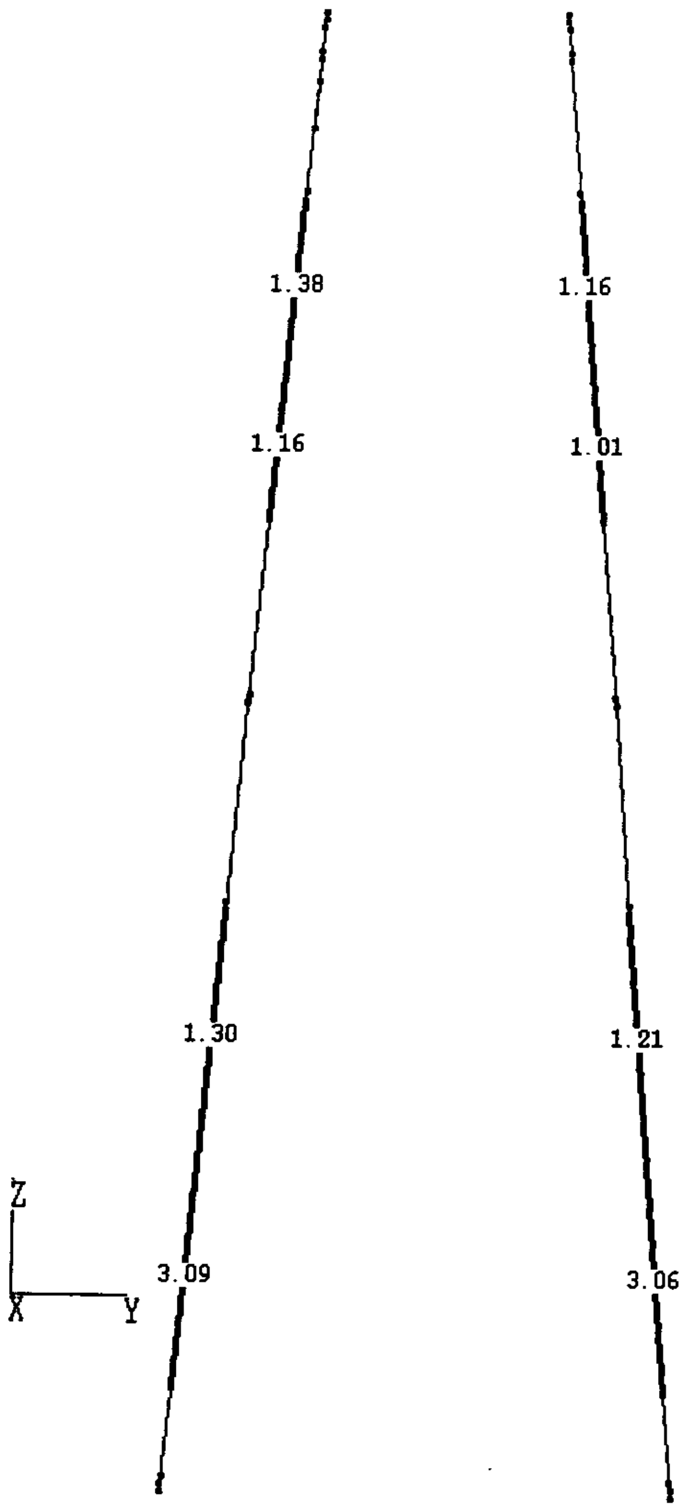


Figure B-6: Benchmark Platform - Column Row A- Pile

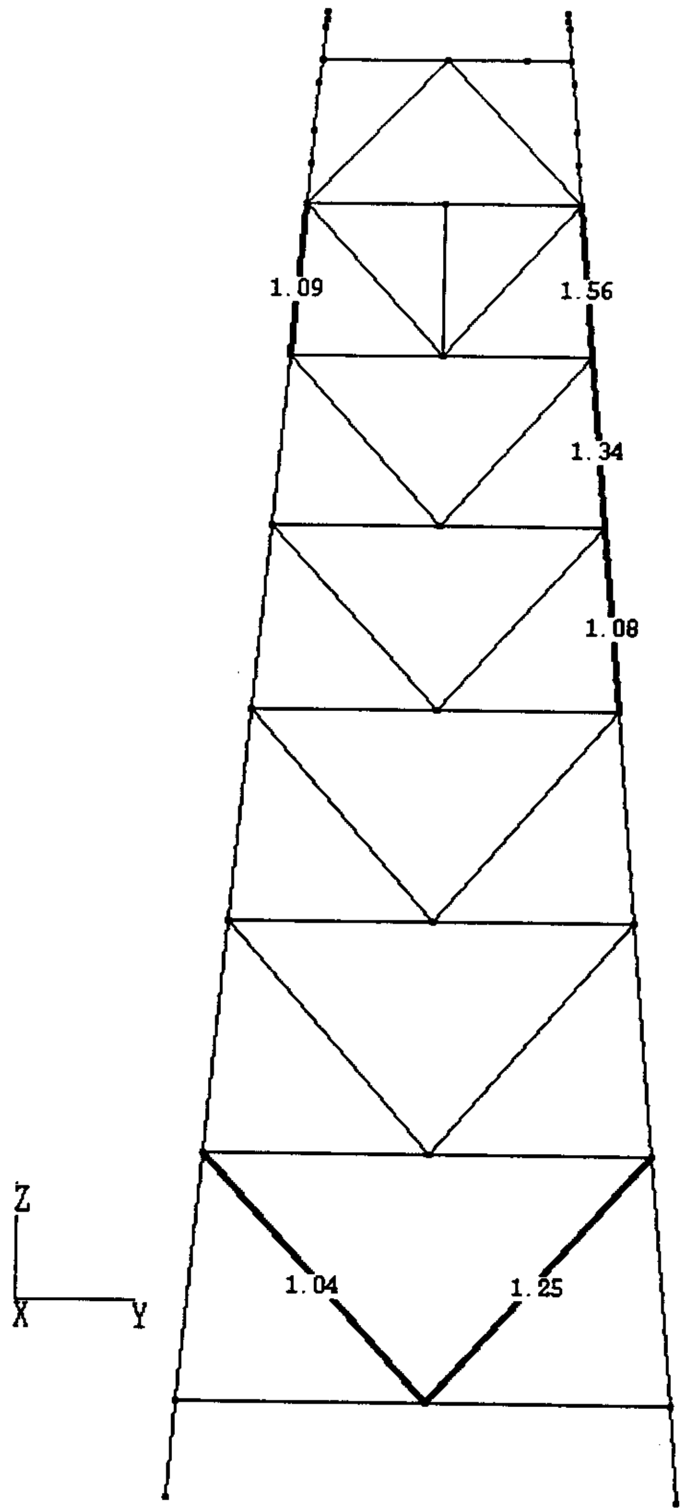


Figure B-7: Benchmark Platform - Column Row B - Jacket

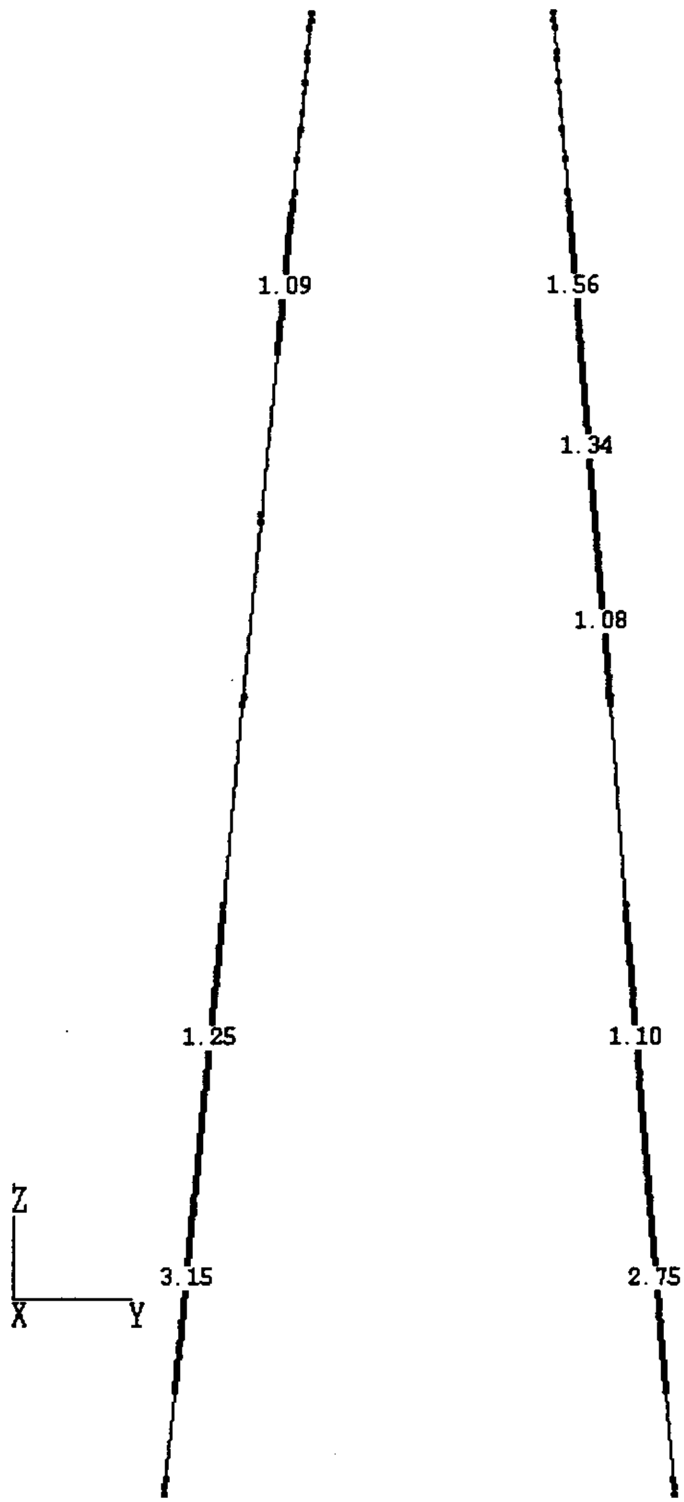


Figure B-8: Benchmark Platform - Column Row B - Pile

PARTICIPANT "H"

1.0 SUMMARY

The following is a summary of the benchmark analysis performed as part of the Joint Industry Project on the Trial Application of the Draft API RP 2A-WSD Procedure for assessment of Existing Platforms.

Based on the results of the analyses, the benchmark platform would not requalify under the Draft API criteria for high consequence platforms. It was found that the ultimate capacity in the diagonal direction could not withstand the environmental loads specified in the DRAFT Section 17 (Ref. 1) for this class of structure. In an actual requalification, a hindcast of the prior exposure during hurricane level events would be recommended to verify the analytical results and to check if the platform could requalify under the prior exposure criteria.

A.1 ENVIRONMENTAL CRITERIA

The platform was analyzed in the three principal directions (diagonal, broadside, and end-on) as illustrated on Figure A.1.1. The sea state parameters along with the resulting environmental loads are summarized on Tables A1.1.1 thru A1.1.5. The reference 100-year environmental parameters and associated loads are based on the specifications of API RP 2A-WSD , 20th Edition (Ref. 2).

Wave loads on the deck were computed using the following equation:

$$F_{Deck} = 1/2\ \rho\ C_d\ A_p\ [\ KF^{*}u_w + V_{IL}\ ]^2$$

where:

	ρ	=	density of water
	C_d	=	drag coefficient (1.5 for diagonal, 2.0 for end-on and broadside)
	A_p	=	projected area of the deck inundated with water
	KF	=	wave kinematics factor, 0.88 for hurricanes
	u_w	=	wave-induced water particle velocity at wave crest
	V_{IL}	=	in-line component of current.

The wave-induced water particle velocities are obtained from 10th-order Stream function wave theory.

A.2 3-D MODEL GENERATION

In modeling the behavior of the platform, two types of analyses were performed. The first was a series of linear elastic analyses using the StruCAD*3D computer program. For the second set of analyses, the non-linear analysis program, KARMA, was used to assess the ultimate capacity of the platform.

The first model was developed using StruCAD*3D. Since this program performs a linear elastic analysis, the main objective was to simulate the behavior of the platform at lower load levels where the member stresses are well below yield. This model was also used to obtain estimates of the joint can capacities consistent with the formulation prescribed in API RP 2A-WSD(Ref. 2) with the safety factors removed.

For the material yield strength, a mean value of 42 ksi was assumed for all jacket, deck, and foundation members as per the requirements of the study. In the absence of any additional information, the nominal member wall thicknesses specified on the drawings were used in the model.

All major appurtenances were modelled to capture their associated wave load but not their stiffness. These include: boat landings, barge bumpers, risers, and conductors. In a more detailed assessment, there may be some merit to investigating the contribution from the conductors to the ultimate capacity of the platform. This would require an assessment of both the conductor capacity, as well as the ability of the conductor guides and associated framing members to transfer these loads.

The environmental loads (wave, current, and wind) were previously described in Section A.1. The loads on the jacket were computed internally by StruCAD*3D. However, since the waves overtop the deck even for waves with a recurrence interval of 100 years, the additional loads on both the deck members and equipment were computed separately following the recommendations in the draft of Section 17 (Ref. 1).

The shear strength profile was provided by a 1969 McClelland Engineers boring. The design profile was based on driven samples 2.25 inches in diameter with emphasis on the minivane test results in the very soft clays and unconfined compression tests when shear strengths exceeded 250 psf. Presently, it is industry practice to base shear strength profiles on unconsolidated undrained (UU) triaxial and minivane tests on pushed samples at least 3.0 inches in diameter. UU tests typically correlate very closely to minivane test results.

A study by Emrich (Ref. 4), expanded upon by McClelland (Ref. 5), showed how sample strength is affected by disturbance (driven versus pushed) and sample tube diameter. The strength ratio of a 3.0 inch pushed sample and a 2.25 inch driven sample was 1.4 for minivane tests. The ratio of unconfined compression tests for the same set of sampling parameters was 1.5. Work by Quiros on three Gulf of Mexico sites (Ref. 6) showed that a unique correlation does not exist for all sites. However, the ratio of pushed versus driven strengths averaged about 1.3. Therefore, the best fit minivane shear strength profile from the site was factored by 1.3 and used as the design soil profile.

The foundation was modelled using non-linear springs to represent the lateral and axial behavior of the piles in the soil. The computer program APILE (Ref. 7) was used to determine the axial soil response. Nonlinear t-z and q-z curves for side friction and end bearing , respectively, were computed at specified depths below the mudline. The criteria of Vijayvergiya (Ref. 8) were selected for the analysis, initially with residual friction of 80% of the maximum value for modelling post-peak behavior . Residual friction created a couple of analysis problems. First, because peak friction and maximum pile capacity are not reached simultaneously in the t-z approach, the resulting capacity (3750 kips) was less than the capacity computed using API 1993 (4250 kips). Second, the post peak curve shape created convergence and instability problems for the KARMA analyses. Therefore, for StruCAD*3D analyses the t-z's were factored to make the t-z pile capacity equivalent to the capacity predicted by API. For KARMA analyses the t-z's were truncated to eliminate the post peak behavior and then factored to obtain the predicted axial capacity.

Lateral soil springs (p-y's) were generated using the program LPILE, Version 3.0 (Ref. 9). For extreme loads in pushover analyses, where deflections will be high and virgin soil will be loaded for the first time, static p-y's are considered appropriate. Since most of the soil profile was clay, Matlock's soft clay criteria were generally used (Ref. 10).

A.3 Software Description

A non-linear analysis model was developed using the KARMA program. This model has the same fundamental assumptions as the StruCAD*3D model described in the previous section. KARMA has the capability to model both the material and geometric non-linearities which will occur at the higher load levels.

The element types were selected based on the expected behavior of each member at its ultimate load through A.4.6. The ultimate strengths reported do not account for the capacity of the joint cans. In fact, a check using the API criteria with the safety factors removed indicated that, although the joint cans could support the 100-year loads, several would fail before attaining the reported ultimate loads. So at those locations, the strengths of the joint cans would be the governing component. As these were not the governing directions, the impact of joint can capacity on the ultimate capacity was not investigated further. However, this issue is not easily addressed. KARMA, as with most non-linear pushover analyses, does not have the capability to explicitly account for the joint can capacity in the ultimate strength analyses. In previous analyses, we have addressed this issue by degrading the member capacities to match the joint can capacities. However, there are various uncertainties with this procedure. First, our experience is that the API joint can capacity formulation is generally conservative even after the safety factor is removed. Second, obviously as the joint cans fail, this will change the internal load distribution. So until the joint can capacity failure and load re-distribution algorithms are incorporated into the pushover analysis program, the simplified procedures for including the effect of joint can failures are at best first pass approximations. We therefore recommend further research in this area which would allow us to incorporate this capability into the ultimate strength analysis programs.

Another important issue is the wave loads impacting the deck. In this analysis we found that the ultimate strength for the broadside and end-on directions could vary significantly depending on how these loads are incremented from the 100-year loads to ultimate failure. In addition, these loads become an increasing component of the total base shear for the higher return periods (Table A1.1.5). Therefore, further validation and calibration of the wave impact load algorithm are also important issues.

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9. Reese, L.C., and Wang, S-T., *"Documentation of Computer Program LPILE. Version 3.0,"* Ensoft, Inc. Users Manual, 1989.

10. Matlock, H., *"Correlations for Design of Laterally Loaded Piles in Soft Clay,"* Offshore Technology Conference Proceeding Paper OTC No. 1204, Houston, Texas, 1970.

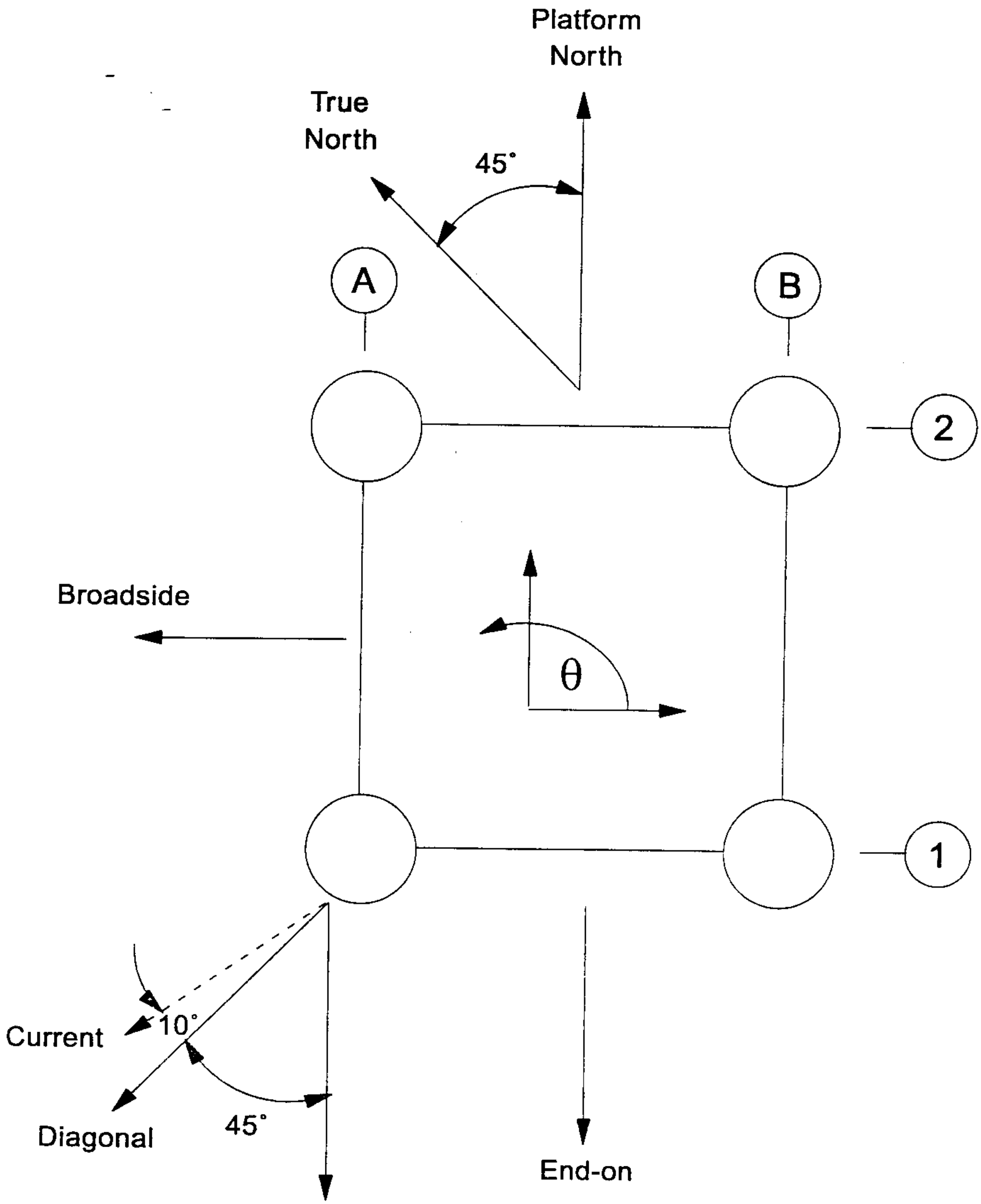


Figure A.1.1
Wave Load Directions

TABLE A1.1.1
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
Oceanographic Criteria for 100 Year Storm (API 20th Edition)

Water Depth:	157.0 ft
Storm Surge:	3.5 ft
Storm Water Depth:	160.5 ft
Omnidirectional Wave Height	63.0 ft

Oceanographic Parameter	End On	Broadside	Diagonal
Wave Directionality Factor	0.90	0.95	1.00
Wave Height (ft)	56.70	59.85	63.00
Wave Direction (degrees)	270.00	180.00	225.00
Wave Period (secs)	13.00	13.00	13.00
Apparent Wave Period (secs)	13.45	13.64	13.76
Wave Kinematics Factor	0.88	0.88	0.88
Wind Speed @ 10m (knots)	80.00	80.00	80.00
Wind Direction (degrees)	270.00	180.00	225.00
Wind Profile	API	API	API
Current (knots)	2.10	2.10	2.10
Effective Current (knots)	1.68	1.68	1.79
Current Direction (degrees)	215.00	215.00	215.00
Current Blockage Factor	0.80	0.80	0.85
Marine Growth (inches)	1.50	1.50	1.50

NOTES:
1. Refer to figure A.1.1 for wave, wind and current directions.
2. Stream Function wave theory used in wave load calculation.
3. Current profile assumed uniform for entire water depth.
4. Stretching of current considered.
5. Marine growth considered from 7ft to 158ft above mudline.
6. Wave heights include wave directionality factor.

TABLE A1.1.2
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
Oceanographic Criteria for Section 17 (API 20th Edition)

Water Depth:	157.0 ft
Storm Surge:	3.0 ft
Storm Water Depth:	160.0 ft
Omnidirectional Wave Height	68.0 ft

Oceanographic Parameter	End On	Broadside	Diagonal
Wave Directionality Factor	0.90	0.95	1.00
Wave Height (ft)	61.20	64.60	68.00
Wave Direction (degrees)	270.00	180.00	225.00
Wave Period (secs)	13.00	13.00	13.00
Apparent Wave Period (secs)	14.00	14.21	14.36
Wave Kinematics Factor	0.88	0.88	0.88
Wind Speed @ 10m (knots)	85.00	85.00	85.00
Wind Direction (degrees)	270.00	180.00	225.00
Wind Profile	API	API	API
Current (knots)	2.30	2.30	2.30
Effective Current (knots)	1.84	1.84	1.96
Current Direction (degrees)	215.00	215.00	215.00
Current Blockage Factor	0.80	0.80	0.85
Marine Growth (inches)	1.50	1.50	1.50

NOTES:

1. Refer to figure A.1.1 for wave, wind and current directions.
2. Stream Function wave theory used in wave load calculation.
3. Current profile assumed uniform for entire water depth.
4. Stretching of current considered.
5. Marine growth considered from 7ft to 158ft above mudline.
6. Wave heights include wave directionality factor.

TABLE A1.1.3
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
Oceanographic Criteria for 500 Year Storm (API 20th Edition)

Water Depth:	157.0 ft
Storm Surge:	4.5 ft
Storm Water Depth:	161.5 ft
Omnidirectional Wave Height	78.0 ft

Oceanographic Parameter	End On	Broadside	Diagonal
Wave Directionality Factor	0.90	0.95	1.00
Wave Height (ft)	70.20	74.10	78.00
Wave Direction (degrees)	270.00	180.00	225.00
Wave Period (secs)	13.00	13.00	13.00
Apparent Wave Period (secs)	13.58	13.81	13.97
Wave Kinematics Factor	0.88	0.88	0.88
Wind Speed @ 10m (knots)	90.00	90.00	90.00
Wind Direction (degrees)	270.00	180.00	225.00
Wind Profile	API	API	API
Current (knots)	2.70	2.70	2.70
Effective Current (knots)	2.16	2.16	2.30
Current Direction (degrees)	215.00	215.00	215.00
Current Blockage Factor	0.80	0.80	0.85
Marine Growth (inches)	1.50	1.50	1.50

NOTES:
1. Refer to figure A.1.1 for wave, wind and current directions.
2. Stream Function wave theory used in wave load calculation.
3. Current profile assumed uniform for entire water depth.
4. Stretching of current considered.
5. Marine growth considered from 7ft to 158ft above mudline.
6. Wave heights include wave directionality factor.

TABLE A1.1.4
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
Oceanographic Criteria for 200 Year Storm (API 20th Edition)

Water Depth:	157.0 ft
Storm Surge:	4.0 ft
Storm Water Depth:	161.0 ft
Omnidirectional Wave Height	69.2 ft

Oceanographic Parameter	End On	Broadside	Diagonal
Wave Directionality Factor	0.90	0.95	1.00
Wave Height (ft)	62.28	65.74	69.20
Wave Direction (degrees)	270.00	180.00	225.00
Wave Period (secs)	13.00	13.00	13.00
Apparent Wave Period (secs)	13.51	13.73	13.87
Wave Kinematics Factor	0.88	0.88	0.88
Wind Speed @ 10m (knots)	85.00	85.00	85.00
Wind Direction (degrees)	270.00	180.00	225.00
Wind Profile	API	API	API
Current (knots)	2.40	2.40	2.40
Effective Current (knots)	1.92	1.92	2.04
Current Direction (degrees)	215.00	215.00	215.00
Current Blockage Factor	0.80	0.80	0.85
Marine Growth (inches)	1.50	1.50	1.50

NOTES:

1. Refer to figure A.1.1 for wave, wind and current directions.
2. Stream Function wave theory used in wave load calculation.
3. Current profile assumed uniform for entire water depth.
4. Stretching of current considered.
5. Marine growth considered from 7ft to 158ft above mudline.
6. Wave heights include wave directionality factor.

TABLE A1.1.5
PMB-JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
ENVIRONMENTAL LOAD SUMMARY

	ENVIRONMENTAL LOADS (kips)			
	100 yr.	**Sect. 17**	**200 yr.**	**500 yr.**
DIAGONAL				
Jacket	2134	2568	2596	3234
Wave Impact on Deck	110	340	530	1690
Wind	74	84	83	78
TOTAL	2318	2992	3209	5002
BROADSIDE				
Jacket	1815	2196	2218	2771
Wave Impact on Deck	85	165	310	970
Wind	82	88	88	99
TOTAL	1982	2449	2616	3840
END-ON				
Jacket	1535	1860	1880	2350
Wave Impact on Deck	60	95	135	670
Wind	74	83	78	88
TOTAL	1669	2038	2093	3108

file: c:\pmb\loads.xls

TABLE A.4.1
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS
SUMMARY OF RESULTS

WAVE DIRECTION	LOAD LEVEL @ UNITY CHECK OF 1.0	100-YR. LOAD	ULTIMATE CAPACITY	RESERVE STRENGTH RATIO
Diagonal	1252	2318	2628	1.13
Broadside	1274	1982	2895	1.46
End-On	1260	1669	2827	1.69

NOTES:
1. All loads are base shears in kips at the mudline elevation.
2. The second column is the lateral load level at which the first component reaches a unity check of 1.0, or the first pile reaches design level per API RP 2A-WSD, 20th Edition, with all safety factors included and the 1/3 increase in allowable stresses.
3. The ultimate capacities for the broadside and end-on directions do not account for the joint can capacities (refer to Section A.4 for further discussion of this issue).

c:\pmb\summary.xls

FIGURE A.4.1
BASE SHEAR vs. DECK DISPLACEMENT
DIAGONAL DIRECTION

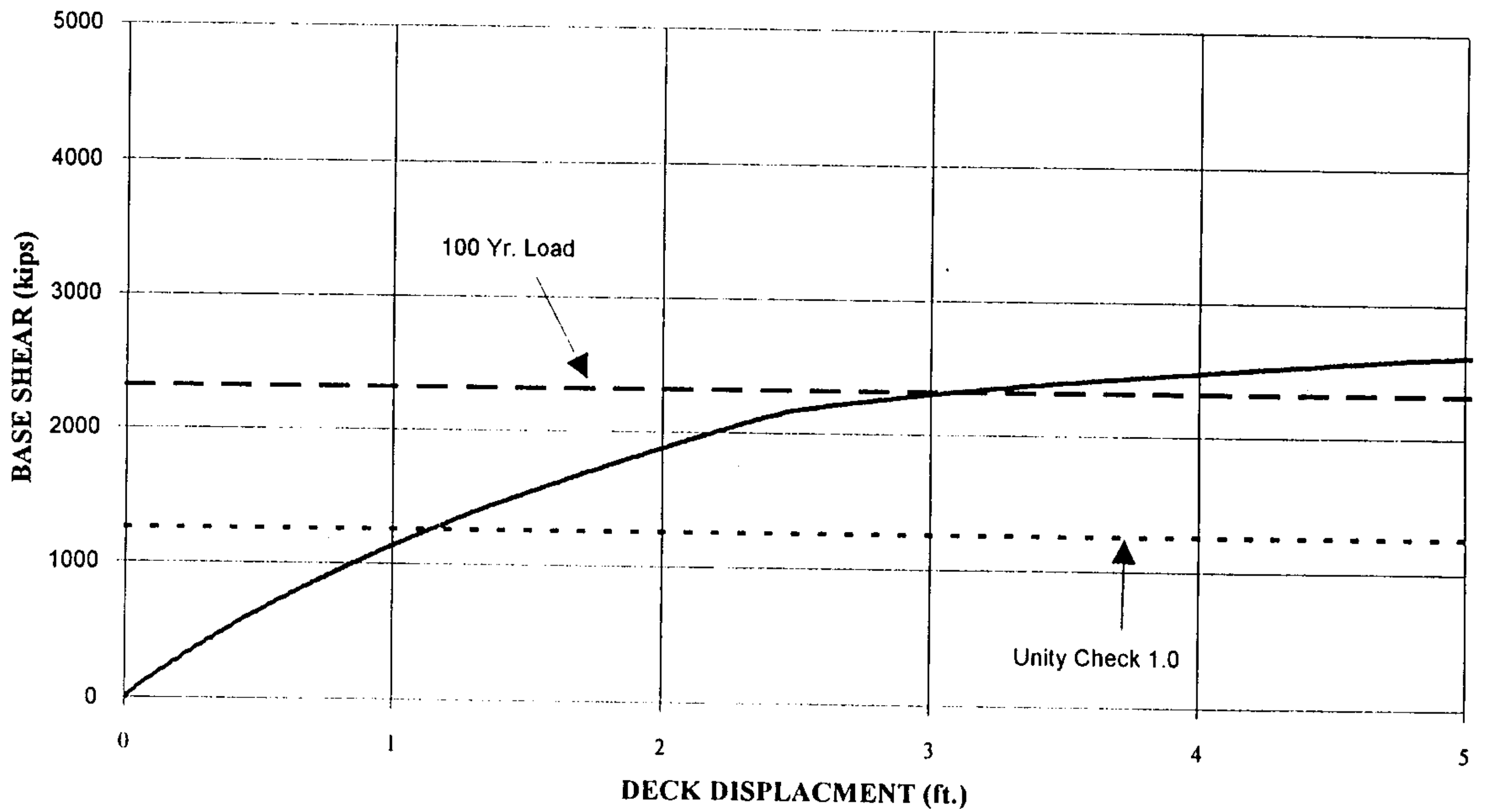


TABLE A.4.2
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS

MEMBER FAILURE SEQUENCE - DIAGONAL DIRECTION

SEQUENCE NO.	MEMBER NO.	BASE SHEAR (KIPS)	MEMBER LOCATION
1	6093-6097	1636	PILE A2 FROM EL. (-)167' TO (-)169'
2	6095-6099	1668	PILE A1 FROM EL. (-)167' TO (-)169'
3	6094-6098	1690	PILE B2 FROM EL. (-)167' TO (-)169'
4	6097-6101	1808	PILE A2 FROM EL. (-)164.5' TO (-)167'
5	6099-6103	1836	PILE A1 FROM EL. (-)164.5' TO (-)167'
6	6098-6102	1863	PILE B2 FROM EL. (-)164.5' TO (-)167'
7	699-799	2117	LEG B2 FROM EL. (-)27' TO (-)8'
8	611-711	2167	LEG A1 FROM EL. (-)27' TO (-)8'
9	695-699	2364	HORZ. BRACE @ EL. (-)27' EAST SIDE
10	611-651	2375	HORZ. BRACE @ EL. (-)27' SOUTH SIDE
11	619-659	2375	HORZ. BRACE @ EL. (-)27' NORTH SIDE
12	611-615	2389	HORZ. BRACE @ EL. (-)27' WEST SIDE
13	659-699	2389	HORZ. BRACE @ EL. (-)27' NORTH SIDE
14	691-695	2428	HORZ. BRACE @ EL. (-)27' EAST SIDE
15	615-619	2463	HORZ. BRACE @ EL. (-)27' WEST SIDE
16	651-691	2477	HORZ. BRACE @ EL. (-)27' SOUTH SIDE
17	2699-2799	2533	PILE B2 FROM EL. (-)27' TO (-)8'
18	511-611	2537	LEG A1 FROM EL. (-)48' TO (-)27'
19	2611-2711	2561	PILE A1 FROM EL. (-)27' TO (-)8'
20	2599-2699	2566	PILE B2 FROM EL. (-)48' TO (-)27'
21	599-699	2583	LEG B2 FROM EL. (-)48' TO (-)27'
22	519-619	2590	LEG A2 FROM EL. (-)48' TO (-)27'
23	591-691	2593	LEG B1 FROM EL. (-)48' TO (-)27'
24	2511-2611	2593	PILE A1 FROM EL. (-)48' TO (-)27'

TABLE A.4.5
ULTIMATE STRENGTH ANALYSIS
LOAD - DEFLECTION DATA
DIAGONAL DIRECTION

LOAD STEP	DISPLACEMENT (ft.)	SHEAR (kips)
1	0.00	0
2	0.10	147
3	0.50	637
4	0.70	846
5	0.80	945
6	0.90	1038
7	1.00	1132
8	1.20	1301
9	1.40	1463
10	1.60	1604
11	1.70	1677
12	1.90	1812
13	2.00	1880
14	2.20	2008
15	2.40	2126
16	2.60	2203
17	2.80	2253
18	3.00	2303
19	3.30	2360
20	3.60	2413
21	3.81	2445
22	4.01	2477
23	4.20	2505
24	4.42	2537
25	4.70	2575
26	4.81	2590
27	4.99	2611
28	5.03	2614
29	5.09	2621
30	5.15	2628

NOTES:
1. Only a representative sample of the load steps that were used to generate the plot are provided above.
2. The lateral displacments are at Deck Level +43' at South East leg.

FIGURE A.4.2
BASE SHEAR vs. DECK DISPLACEMENT
BROADSIDE DIRECTION

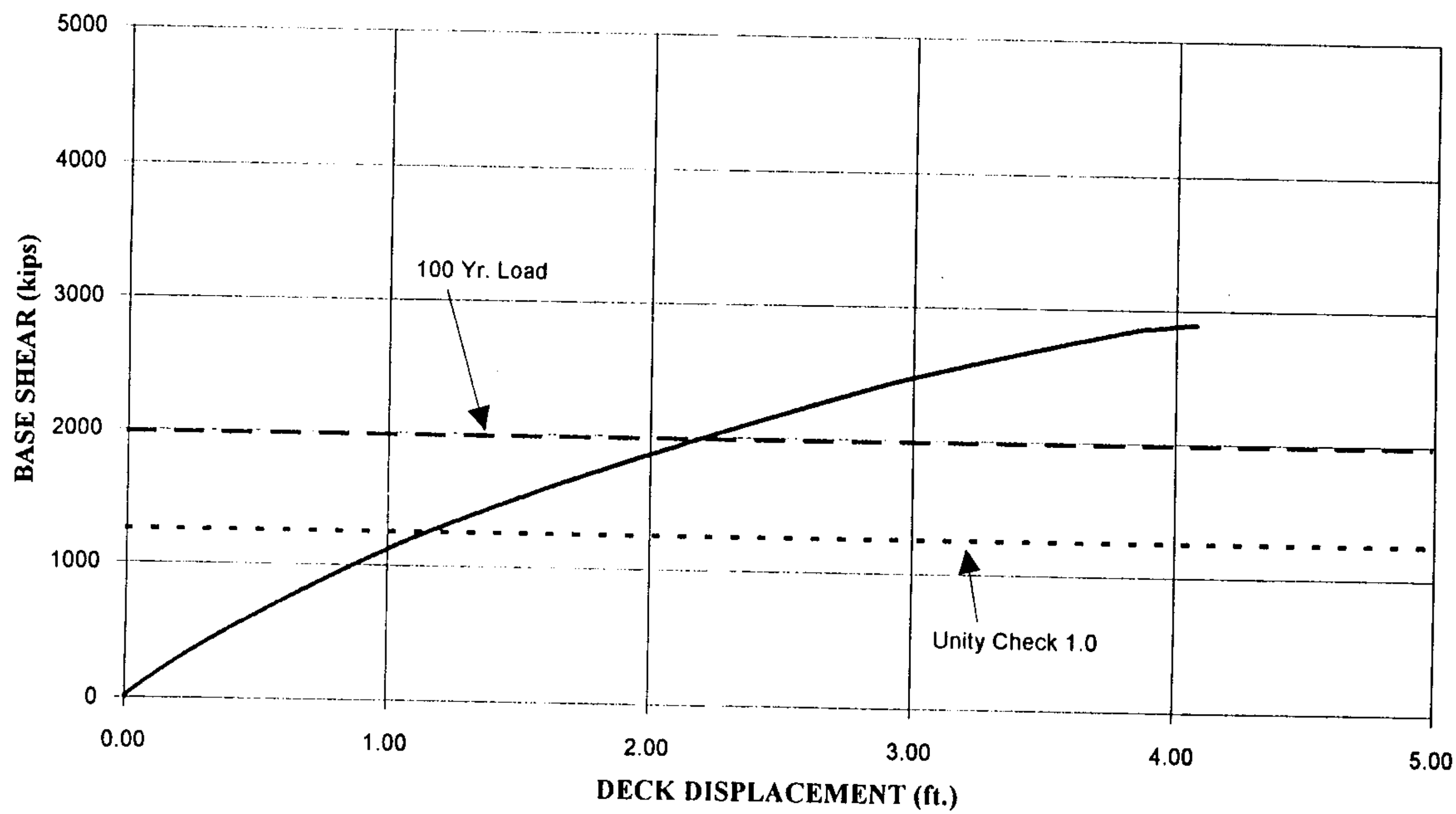


TABLE A.4.3
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS

MEMBER FAILURE SEQUENCE - BROADSIDE DIRECTION

SEQUENCE NO.	MEMBER NO.	BASE SHEAR (kips)	MEMBER LOCATION
1	6093-6097	1554	PILE A2 FROM EL. (-)167' TO (-)169'
2	6094-6098	1585	PILE B2 FROM EL. (-)167' TO (-)169'
3	6095-6099	1655	PILE A1 FROM EL. (-)167' TO (-)169'
4	6097-6101	1722	PILE A2 FROM EL. (-)164.5' TO (-)167'
5	6098-6102	1757	PILE B2 FROM EL. (-)164.5' TO (-)167'
6	6099-6103	1827	PILE A1 FROM EL. (-)164.5' TO (-)167'
7	1095-1195	2306	COL. BETWEEN EL. (+)33' TO (+)43' EAST SIDE
8	1015-1115	2315	COL. BETWEEN EL. (+)33' TO (+)43' WEST SIDE
9	1015-1017	2691	COL. BETWEEN EL. (+)33' TO (+)43' WEST SIDE
10	1095-1097	2694	COL. BETWEEN EL. (+)33' TO (+)43' EAST SIDE
11	1013-1015	2702	COL. BETWEEN EL. (+)33' TO (+)43' WEST SIDE
12	1093-1095	2705	COL. BETWEEN EL. (+)33' TO (+)43' EAST SIDE
13	1091-1093	2788	HORZ. BRACE @ EL. (+)33' EAST SIDE
14	1011-1013	2796	HORZ. BRACE @ EL. (+)33' SOUTH SIDE
15	6108-2191	2821	PILE B1 FROM EL. (-)162' TO (-)157'
16	6089-6093	2823	PILE A2 FROM EL. (-)169' TO (-)172'
17	6105-2119	2826	PILE A2 FROM EL. (-)162' TO (-)157'
18	1017-1019	2870	HORZ. BRACE @ EL. (+)33' WEST SIDE
19	1097-1099	2870	HORZ. BRACE @ EL. (+)33' EAST SIDE
20	6092-6096	2879	PILE B1 FROM EL. (-)169' TO (-)172'

TABLE A.4.6
ULTIMATE STRENGTH ANALYSIS
LOAD - DEFLECTION DATA
BROADSIDE DIRECTION

LOAD STEP	DISPLACEMENT (ft.)	SHEAR (kips)
1	0.00	0
2	0.01	28
3	0.25	349
4	0.50	627
5	0.70	831
6	0.90	1025
7	1.00	1115
8	1.10	1206
9	1.20	1284
10	1.30	1362
11	1.40	1440
12	1.50	1511
13	1.60	1585
14	1.70	1655
15	1.80	1725
16	1.90	1796
17	2.00	1858
18	2.20	1989
19	2.40	2116
20	2.60	2239
21	2.80	2356
22	3.00	2468
23	3.40	2655
24	3.60	2746
25	3.70	2788
26	3.80	2829
27	3.90	2862
28	4.00	2881
29	4.04	2890
30	4.08	2895

NOTES:

1. Only a representative sample of the load steps that were used to generate the plot are provided above.
2. The lateral displacments are at Deck Level +43' at South East leg.

FIGURE A.4.3
BASE SHEAR vs. DECK DISPLACEMENT
END-ON DIRECTION

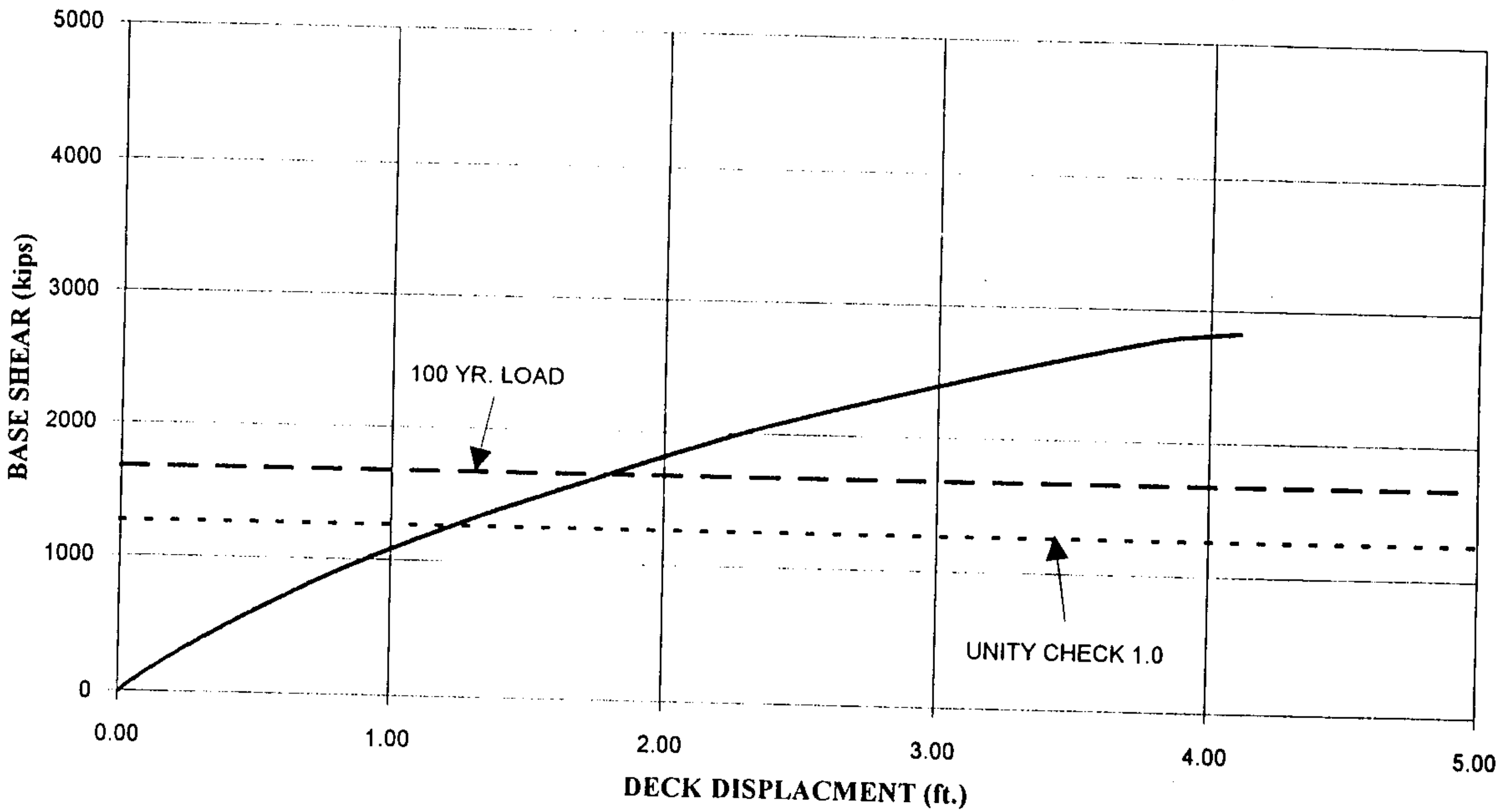


TABLE A.4.4
PMB - JOINT INDUSTRY PROJECT
BENCHMARK ANALYSIS

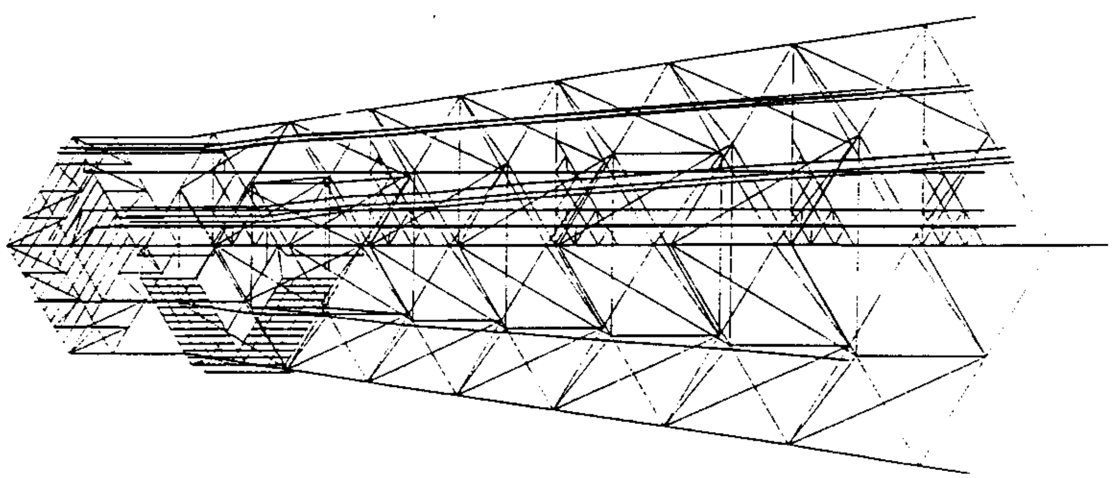
MEMBER FAILURE SEQUENCE - END ON DIRECTION

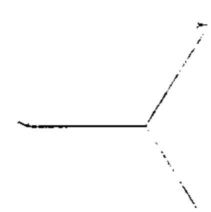
SEQUENCE NO.	MEMBER NO.	BASE SHEAR (kips)	MEMBER LOCATION
1	6095-6099	1630	PILE A1 FROM EL. (-)167' TO (-)169'
2	6094-6098	1646	PILE B2 FROM EL. (-)167' TO (-)169'
3	6093-6097	1662	PILE A2 FROM EL. (-)167' TO (-)169'
4	6099-6103	1800	PILE A1 FROM EL. (-)164.5' TO (-)167'
5	6098-6102	1822	PILE B2 FROM EL. (-)164.5' TO (-)167'
6	6097-6101	1836	PILE A2 FROM EL. (-)164.5' TO (-)167'
7	1051-1071	2634	HORZ. BRACE @ EL. (+)33' SOUTH SIDE
8	1059-1079	2640	HORZ. BRACE @ EL. (+)33' NORTH SIDE
9	1031-1051	2651	HORZ. BRACE @ EL. (+)33' SOUTH SIDE
10	1039-1059	2651	HORZ. BRACE @ EL. (+)33' NORTH SIDE
11	6108-2191	2702	PILE B1 FROM EL. (-)162' TO (-)157'
12	1071-1091	2709	HORZ. BRACE @ EL. (+)33' SOUTH SIDE
13	1019-1039	2719	HORZ. BRACE @ EL. (+)33' NORTH SIDE
14	1011-1031	2724	HORZ. BRACE @ EL. (+)33' SOUTH SIDE
15	1079-1099	2734	HORZ. BRACE @ EL. (+)33' NORTH SIDE
16	1051-1105	2739	COL. BETWEEN EL. (+)33' TO (+)43' SOUTH SIDE
17	1059-1125	2747	COL. BETWEEN EL. (+)33' TO (+)43' NORTH SIDE
18	6092-6096	2795	PILE B1 FROM EL. (-)169' TO (-)172'

TABLE A.4.7
ULTIMATE STRENGTH ANALYSIS
LOAD - DEFLECTION DATA
END-ON DIRECTION

LOAD STEP	DISPLACEMENT (ft.)	SHEAR (kips)
1	0.00	0
2	0.10	143
3	0.20	262
4	0.30	381
5	0.40	499
6	0.50	599
7	0.60	700
8	0.70	802
9	0.80	900
10	0.90	993
11	1.00	1077
12	1.20	1237
13	1.40	1389
14	1.60	1533
15	1.80	1671
16	2.00	1811
17	2.20	1944
18	2.40	2067
19	2.60	2177
20	2.80	2284
21	3.00	2387
22	3.20	2488
23	3.40	2580
24	3.60	2672
25	3.80	2758
26	4.00	2812
27	4.02	2814
28	4.04	2818
29	4.09	2825
30	4.11	2827

NOTES:
1. Only a representative sample of the load steps that were used to generate the plot are provided above.
2. The lateral displacments are at Deck Level +43' at South East leg.





PARTICIPANT "I"

1.0 Summary

API Task Group 92-5 has developed a draft version of API RP 2A Section 17.0 – Assessment of Existing Platforms. This is a product of collective oil industry expertise with contribution mainly by the members of API Task Group 92-5 as well as other API members. A joint industry project (JIP) was proposed by PMB Engineering of San Francisco, California for the purpose of testing the methodology presented in the draft recommendations. This program is sponsored by the Minerals Management Services (MMS) with the participants mainly from oil companies (or operators) and consulting engineering companies.

A Benchmark platform is selected by PMB and provided to the participants to perform the ultimate strength analysis based on the provision of API RP 2A section 17.0 (draft). The
Benchmark platform (a 4-leg, ungrouted, P/D platform with 4 wells) is located at Ship Shoal Block 220 of Gulf of Mexico in 157 ft of water. The structural drawings, design loads and soil data are provided by PMB.

As one of the JIP Benchmark participants,
. Company, has performed the ultimate strength analysis of the Benchmark platform using MicroSAS, an in-house developed computer program mainly for offshore platforms design and analysis.

Four wave directions (205, 180, 225 and 270 deg) are considered in the static push-over analysis. First, the vertical loads are applied the structural system. Secondly, the lateral load (wave, current and wind) are applied to the structural system in increment with each load step corresponding to a fraction (multiplied) of the ultimate lateral load.

The ultimate load (push-over load level) is calculated for each wave direction and the results are expressed in terms of reserve strength ratio (RSR).The reserve strength ratio (RSR) is defined as the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present API RP 2A procedures.

The results of the ultimate strength analysis (static push-over analysis) showed that the reserve strength ratios (RSR) of the Benchmark platform are in the range of 0.99 to 1.56 in the four wave directions (205, 180, 225 and 270 deg.) applied. The lowest reserve strength ratio of this platform is 0.99 which occurred in the wave direction of 205 deg.(MicroSAS convention). This is based on the comparison with the reference lateral load of the current API RP 2A 20th edition.

In addition, the load-deflection plots are made for four wave directions. The deck joint #2011 which is located at EL.(+)43' - 0 of deck leg (B-1), south-east corner with respect to platform north, is selected as a representative joint in reference to the deck displacements. Due to the weak soil skin resistance (axial), the axial displacements of the piles are very significant prior to reaching the ultimate lateral load level, especially in the diagonal wave directions (205, 225 deg.). The structural global response and local member response (buckled or yielded) are properly documented in the report.

It should be pointed out that the reserve strength ratio is directional dependent. It raised the question about how many wave directions should be considered in the platform assessment to ensure that the platform's reserve strength is properly evaluated.

Finally, it should be noted that no fatigue analysis was performed in this study. In the meantime, in this analysis, it is assumed that the major joints in the structure have sufficient joint capacity to withstand the loads developed in the static push-over analysis. The joint check and tube member check results for the ultimate strength analysis are discussed in the report.

A.1 Data for Environmental Loads (Ultimate Strength Analysis) :

Number of approach directions 4

1. Orientation with respect to Platform North 245 deg. (clockwise)

(205 deg. MicroSAS)

Wave height	67.50 ft.
Wave Period	13.5 sec. (Tapp = 14.34 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stoke 5th Order Waves

2. Orientation with respect to Platform North 270 deg. (clockwise)

(180 deg. MicroSAS)

Wave height	65.61 ft.
Wave Period	13.5 sec. (Tapp = 14.18 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stoke 5th Order Waves

3. Orientation with respect to Platform North 225 deg. (clockwise)

(225 deg. MicroSAS)

Wave height	64.53 ft.
Wave Period	13.5 sec. (Tapp = 14.36 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stoke 5th Order Waves

4. Orientation with respect to Platform North 180 deg. (clockwise)

(270 deg. MicroSAS)

Wave height	56.23 ft.
Wave Period	13.5 sec. (Tapp = 14.05 sec.)
Current Profile	3.88 ft / sec Constant (from mudline to 159.5' above mudline)
Current Direction	Constant (231 deg. from platform north, clockwise)
Storm Surge	2.5 ft.
Wind Speed @ 10m above msl	143.47 ft / sec (97.82 mph)
Wave Theory	Stoke 5th Order Waves

A.2 3-D Model Generation :

The structural model is a three dimensional model consisting of 444 Nodes and 744 Elements. The Element types are BEAM-COLUMN, STRUT and LINEAR BEAM. The jacket leg and piles are modeled as BEAM-COLUMN elements, vertical diagonal bracing (K-braces), horizontal members around the framing perimeter at each framing level and some major diagonal members in the horizontal framing are model as STRUT elements, the rest of structural members are modeled as LINEAR BEAM elements. See attached sketch of the structural model.

The formulations of BEAM-COLUMN and STRUT elements are similar to that used in the INTRA computer program (Refer to INTRA theoretical manual)

A.3 Software Description : (See Attachment)

Description of Nonlinear solution capabilities within MicroSAS

NONLINEAR is one of several modules that are present in the structural analysis and design program, MicroSAS.

The module NONLINEAR is used to perform incremental Nonlinear Static Analysis using beam, strut and axial spring elements with nonlinear material behavior.

A three-dimensional beam column element is available which can model both geometric and material nonlinearities. Geometric nonlinearities can be modeled either by using a geometric stiffness matrix or via stability functions. Material nonlinearities are modeled using a concentrated plasticity formulation. A parallel plastic hinge model is used to allow the formation of plastic hinges at the element ends. Numerical integration is used to obtain the force-resultant vectors. Material loading-unloading from the yield surface and elastic-perfectly-plastic as well elastic-strain-hardening behavior can be modeled. Both joint and member loads can be accommodated.

Different material behaviors in tension and in compression can be specified for the strut elements. Similarly, detailed material behavior can also be specified for the nonlinear springs.

Large displacement, small strain assumptions and an Updated Lagrangian formulation together with automated incremental-iterative solution schemes are used to trace the global structural response. The algorithms are capable of following the loading as well as the post-collapse unloading path beyond the limit point.

The equilibrium iterations at each load step are performed using Newton-Raphson iterations and accelerated using 'arc length' procedures.

A.4 Ultimate Strength Analysis (Required)

A.4.1 General

Four wave directions are considered in the static push-over analysis. They are 205, 180, 225 and 270 deg. in MicroSAS convention. First, the vertical loads are applied the structural system. Secondly, the lateral load (wave, current and wind) are applied to the structural system in increment with each load step corresponding to a fraction (multiplied) of the ultimate lateral load. The vertical loads include structural weight, deck dead and live loads, conductor weight, buoyancy etc.
It should be pointed out that no fatigue analysis was performed in this study. Furthermore, in this analysis, it is assumed that the major joints in the structure have sufficient joint capacity to withstand the loads developed in the static push-over analysis.
For Ultimate Strength Analysis, the wave load are generated using the MicroSAS program. The wave height (H_W) is varied with the wave directions. The current blockage factor, conductor shielding factor and the wave kinematics factor (0.88) are properly taken into consideration. Drag and inertia coefficients are taken into account the surface effects of tubular members (smooth or rough). The marine growth is also modeled per API RP 2A 20th edition.

The design gravity loads used in this study are summarized as follows:

Buoyancy:	694.76 kips
Gravity Load:	1,035.28 kips (including 5% contingency)
Deck Live Load:	1,425.00 kips
Deck Dead Load:	350.00 kips

A.4.2 Pushover Load Level

The pushover load level for four wave directions considered is summarized as follows :

Wave Direction (deg.)	Base Ultimate Lateral Load (Wind, Wave & Current) (kips)	Pushover Lateral Load (kips)
205	3,120	2,496
180	2,755	2,618
225	2,951	2,361
270	1,992	2,490

A.4.3 Load - Deflection Plots

The load-deflection plots are made for four wave directions (205, 180, 225, and 270 deg.). The deck joint #2011 which is located at EL.(+)43' - 0 of deck leg (B-1), south-east corner with respect to platform north, is selected as a representative joint in reference to the deck displacements.

Two types of load-deflection plot are generated. One is the lateral load versus deck displacement. The other one is the ultimate lateral load factor versus deck displacement. The base ultimate lateral load as shown above is corresponding to the ultimate lateral load factor = 1.0. See attached load-deflection plots for further detail.

It can be seen that the ultimate lateral load factor of the wave direction of 205 deg. is approximate 0.80. In the wave direction of 180 deg., the ultimate lateral load factor is approximate 0.95. While in the wave direction of 225 deg. the ultimate lateral factor is approximate 0.80. In the wave direction of 270 deg., the ultimate lateral load factor is approximate 1.25.

A.4.4 Load Level at Which First Component Reaches IR = 1.0

The load level at which first component reach IR = 1.0 is defined as the first member buckled in the static pushover analysis in the following discussion:

i) Wave Direction 205 deg. (MicroSAS)

The lateral load was increased to a load factor of **0.60** (load step #3 in the static pushover analysis), in that particular snap shot, the member #2204 (16" diameter x 0.375" wall thickness), a horizontal diagonal member at Elevation (-) 157'- 0" is buckled first, which is corresponding to the lateral load level of **1,872 kips.**

ii) Wave Direction 180 deg. (MicroSAS)

The lateral load was increased to a load factor of **0.88** (load step #11 in the static pushover analysis), in that particular snap shot, the member #2204 (16" diameter x 0.375" wall thickness), a horizontal diagonal member at Elevation (-) 157'- 0" is buckled first, which is corresponding to the lateral load level of **2,425 kips.**

iii) Wave Direction 225 deg. (MicroSAS)

The lateral load was increased to a load factor of **0.60** (load step #3 in the static pushover analysis), in that particular snap shot, the member #2204 (16" diameter x 0.375" wall thickness), a horizontal diagonal member at Elevation (-) 157'- 0" is buckled first, which is corresponding to the lateral load level of **1,770 kips.**

iv) Wave Direction 270 deg. (MicroSAS)

The lateral load was increased to a load factor of **0.75** (load step #5 in the static pushover analysis), in that particular snap shot, the member #2204 (16" diameter x 0.375" wall thickness), a horizontal diagonal member at Elevation (-) 157'- 0" is buckled first, which is corresponding to the lateral load level of **1,494 kips.**

It should be pointed out that due to weak soil friction resistance (axial), the axial displacements of the piles are very significant prior to reaching the ultimate lateral load level especially in the diagonal wave directions (205, 225 deg.).

A 4.5 Reference Level Load (API RP 2A 20th Edition, 100 Year Return Period)

The reference level load is based on API RP 2A 20th edition, the design wave with 100-year of return period. The reference level load due to wave and current effects is summarized as follows :

Design wave height: 63 ft.

Design wave period: 13.0 sec (Apparent wave periods: varied)

Wave Direction (deg.)	Wave Height (ft.)	Lateral Load (Wave & Current) (kips)
205	63.00	2,527
180	61.24	2,227
225	60.23	2,367
270	52.48	1,600

The maximum lateral load is 2,527 kips in the wave direction of 205 deg.

A.4.6 Reserve Strength Ratio

The reserve strength ratio (RSR) is defined as the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present API RP 2A procedures.

In this study, a static push-over analysis is used to calculate the ultimate lateral load carrying capacity of the platform for each wave direction considered. It depends on the structural model (different types of finite elements, such as Strut, Beam-Column, Beam, etc.) used in the nonlinear analysis (static push-over analysis), numerical solutions scheme implemented in the computer program, and tolerance of unbalanced forces and displacements specified in the nonlinear analysis, the ultimate strength of the platform might be varied, especially in the local response of the platform. But overall, the trend of global response of the platform should be similar, if the same guidelines, such as Section 17.0 (draft) of API RP 2A are used.

The following is the summary of the reserve strength ratio of the platform being studied :

Wave Direction (deg.)	Ultimate Lateral Load (kips)	Reference Lateral Load (kips)	Reserve Strength Ratio (RSR)
205	2,496	2,527	0.99
180	2,618	2,227	1.18
225	2,361	2,367	1.00
270	2,490	1,600	1.56

From the results shown above, the range of the reserve strength ratios is from 0.99 to 1.56 in four wave directions (205, 180, 225 and 270 deg.) being considered. The lowest reserve strength ratio of this platform is 0.99, and in the wave direction of 205 deg. This is based on the comparison with the reference lateral load of the current API RP 2A 20th edition.

It should be pointed out that the reserve strength ratio is directional dependent. It raised the question about how many wave directions should be considered in the platform assessment to ensure that the platform's reserve strength is properly evaluated.

A.4.7 Ultimate Strength Analysis Results Summary

i)	Wave Direction (MicroSAS) :	205 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	1,872 kips
	Reference Level Load (Sref):	2,527 kips
	Ultimate Capacity (Ru):	2,496 kips
	Reserve Strength Ratio (RSR):	0.99
	Platform Failure Mode:	Jacket
ii)	Wave Direction (MicroSAS):	180 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	2,424 kips
	Reference Level Load (Sref):	2,227 kips
	Ultimate Capacity (Ru):	2,618 kips
	Reserve Strength Ratio (RSR):	1.18
	Platform Failure Mode:	Jacket
iii)	Wave Direction (MicroSAS) :	225 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	1,770 kips
	Reference Level Load (Sref):	2,367 kips
	Ultimate Capacity (Ru):	2,361 kips
	Reserve Strength Ratio (RSR):	1.00
	Platform Failure Mode:	Jacket
iv)	Wave Direction (MicroSAS) :	270 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	1,494 kips
	Reference Level Load (Sref):	1,600 kips
	Ultimate Capacity (Ru):	2,490 kips
	Reserve Strength Ratio (RSR):	1.56
	Platform Failure Mode:	

BENCHMARK PLATFORM ASSESSMENT (SHIP SHOAL BLOCK 220)					8/25/94
ULTIMATE STRENGTH	ANALYSIS RESULTS				
ANALYSIS CASE	3D MODEL				
Wave Direction	205 deg.				
Lateral load level	for first member	with unity check	1.00	(Mem. #2204	,horizontal diagonal)
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck JOINT #2011				
	(In.)	(kips)			
1	4.84	624.08			
2	10.40	1248.15			
3	17.99	1872.23	Mem #2204	Buckled	Horizontal, Diagonal
4	26.40	2184.27	See Note 1		
5	28.62	2246.67	See Note 1		
6	34.46	2340.29	See Note 1		
7	78.11	2402.69	See Note 1		
8	182.38	2496.30	See Note 1		
NOTE 1 :					
	Member Buckled or Yield				
Step 3	#2204	(Buckled)			
Step 4	#2204	(Buckled)			
Step 5	#2204	(Buckled)			
Step 6	#2204, #366	(Buckled)			
	#133	(Yield)			
Srep7	#2204, #366	(Buckled)			
	#133	(Yield)			
Step 8	#2204, #366	(Buckled)			
	#133, #106, #128, #157, #186		(Yield)		

Wave Direction	180 deg.				
Lateral load level	for first member	with unity check	1.00	(Mem. #2204	,horizontal diagonal)
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck JOINT #2011				
	(in.)	(kips)			
1	4.22	551.05			
2	8.89	1102.10			
3	14.08	1653.16			
4	16.93	1928.68			
5	17.53	1983.79			
6	18.46	2066.45			
7	19.09	2121.55			
8	20.03	2204.21			
9	21.35	2314.42			
10	22.14	2369.52			
11	25.10	2424.63	Mem #2204	Buckled	Horizontal, Diagonal
12	26.54	2479.73	See Note 2		
13	38.09	2534.84	See Note 2		
14	89.26	2617.50	See Note 2		
NOTE 2 :					
	Member Buckled or Yield				
Step 11	#2204	(Buckled)			
Step 12	#2204	(Buckled)			
Step 13	#2204, #366	(Buckled)			
Step 14	#2204, #366	(Buckled)			
	#108, #186, #364	(Yield)			

Wave Direction	225 deg.				
Lateral load level	for first member	with unity check	1.00	(Mem. #2204	,horizontal diagonal)
Load Step	Lateral Displacement at Deck JOINT #2011 (in.)	Lateral Load (kips)	Element Failures	Component Failure Mode	Remark
1	4.56	590.15			
2	9.80	1180.29			
3	17.40	1770.44	Mem #2204	Buckled	Horizontal, Diagonal
4	24.96	2065.51	See Note 3		
5	27.03	2124.53	See Note 3		
6	30.16	2213.05	See Note 3		
7	32.90	2272.06	See Note 3		
8	110.71	2360.58	See Note 3		
NOTE 3 :					
	Member Buckled or Yield				
Step 3	#2204	(Buckled)			
Step 4	#2204	(Buckled)			
Step 5	#2204	(Buckled)			
Step 6	#2204	(Buckled)			
Step 7	#2204	(Buckled)			
	#133	(Yield)			
Step 8	#2204, #366	(Buckled)			
	#133, #186	(Yield)			

Wave Direction	270 deg.				
Lateral load level	for first member	with unity check	1.00	(Mem. #2204	,horizontal diagonal)
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck JOINT #2011				
	(in.)	(kips)			
1	3.03	398.42			
2	6.17	796.85			
3	9.58	1195.27			
4	11.38	1394.48			
5	13.02	1494.09	Mem #2204	Buckled	Horizontal, Diagonal
6	14.15	1593.70	See Note 4		
7	15.29	1693.30	See Note 4		
8	17.25	1792.91	See Note 4		
9	18.62	1892.51	See Note 4		
10	20.09	1992.12	See Note 4		
11	21.58	2091.73	See Note 4		
12	23.12	2191.33	See Note 4		
13	26.05	2290.94	See Note 4		
14	29.08	2390.54	See Note 4		
15	39.31	2490.15	See Note 4		
NOTE 4 :					
	Member Buckled or Yield				
Step 5	#2204	(Buckled)			
Step 6	#2204	(Buckled)			
Step 7	#2204	(Buckled)			
Step 8	#2204, #364	(Buckled)			
Step 9	#2204, #364	(Buckled)			
Step 10	#2204, #364	(Buckled)			
Step 11	#2204, #364	(Buckled)			
Step 12	#2204, #364	(Buckled)			
Step 13	#2204, #364	(Buckled)			
Step 14	#2204, #364	(Buckled)			
Step 15	#2204, #364	(Buckled)			

BENCHMARK	PLATFORM							8/24/94
SUMMARY	OF	MEMBER	CAPACITIES	CALCULATED	BY	MICROSAS	PROGRAM	FOR
MEMBERS	BUCKLED	OR	YIELDED	IN	THE	STATIC	PUSH-OVER	ANALYSIS
UNITS	INCHES	KIPS	KIPM	RAD				
NONLINEAR	BEAM	PROPERTIES	FOR	GROUP	GRPBMCOL	(BEAM-COLUM	ELEMENTS)	
MEMBER	P-BUCKLING	P-YIELD	MY-YIELD	MZ-YIELD				
133	2369.15	2572.96	31942.75	31942.75				
106	5047.65	5731.44	63424.38	63424.38				
128	4901.94	5731.44	63424.38	63424.38				
157	4901.94	5731.44	63424.38	63424.38				
186	4901.94	5731.44	63424.38	63424.38				
UNITS	INCHES	KIPS	KIPM	RAD				
NONLINEAR	STRUT	PROPERTIES	FOR	GROUP	EX-HORZ	(STRUT	ELEMENTS)	
MEMBER	P-YIELD	P-BUCKLING	P-ULTIMATE	K-BUCKLING	K-ULTIMATE	U-ELASTIC	U-BUCKLING	
364	773.13	710.23	440.87	2149.64	32.24	0.330393	0.455697	
366	773.13	591.33	309.39	1104.63	16.57	0.535318	0.79055	
2204	773.13	710.23	440.87	2149.64	32.24	0.330393	0.455697	

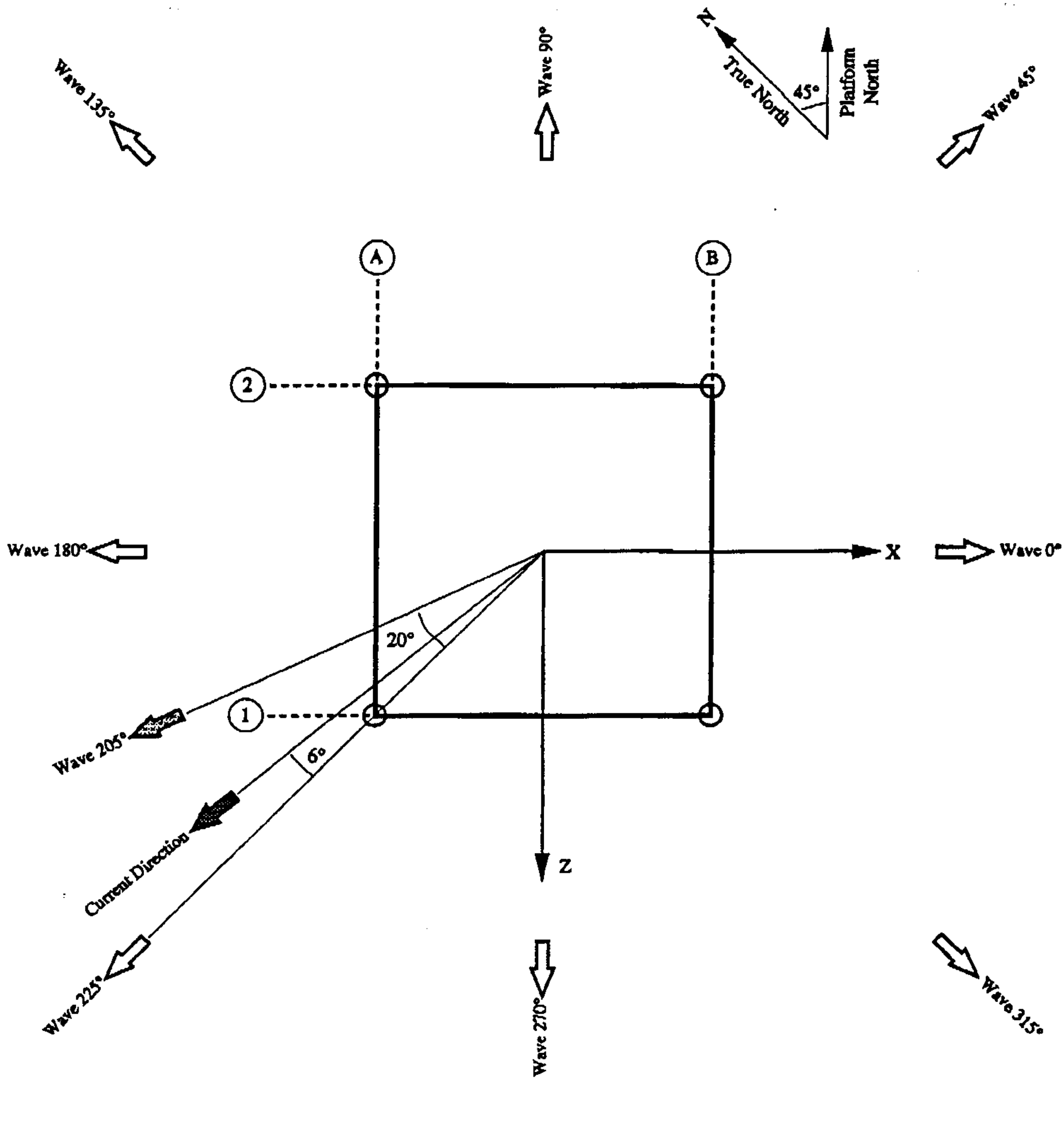


FIG. 1 **Platform Orientation & Wave Direction**

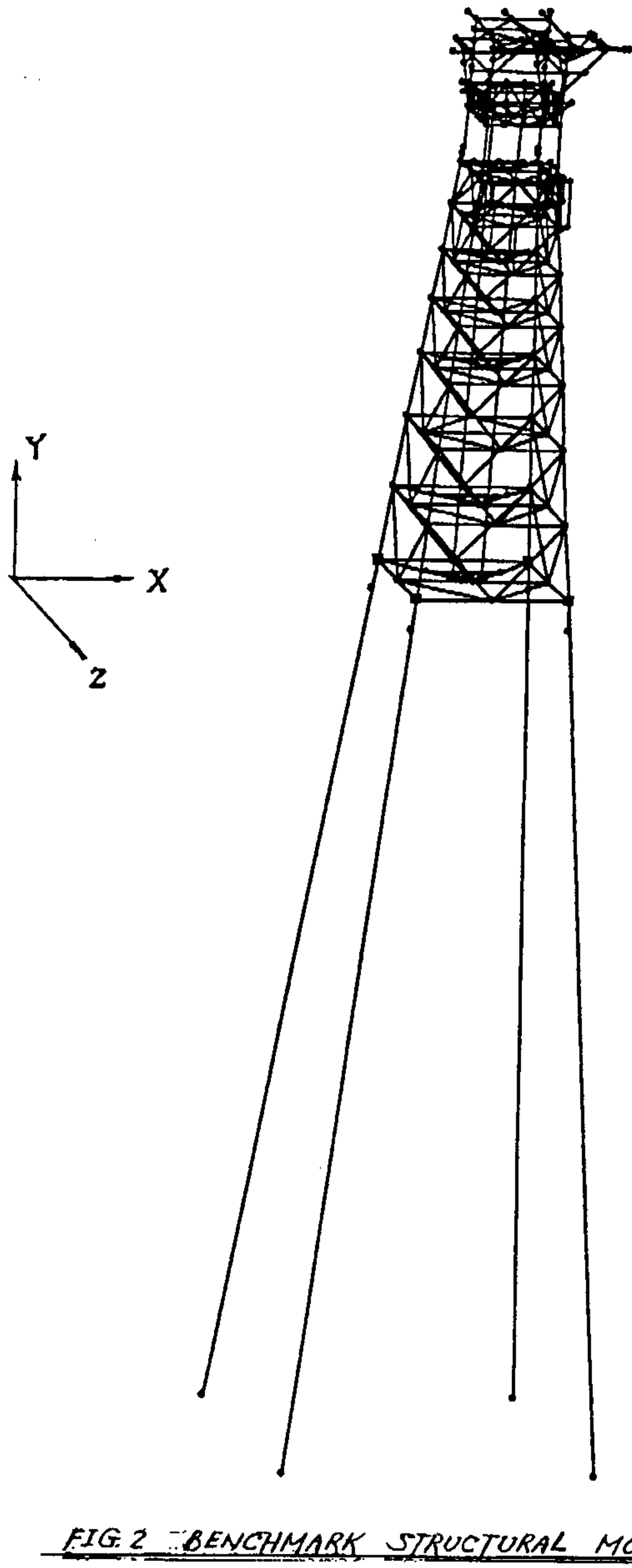


FIG. 2 - BENCHMARK STRUCTURAL MODEL

BENCHMARK PLATFORM ASSESSMENT (SIGNIFICANT ENV.)

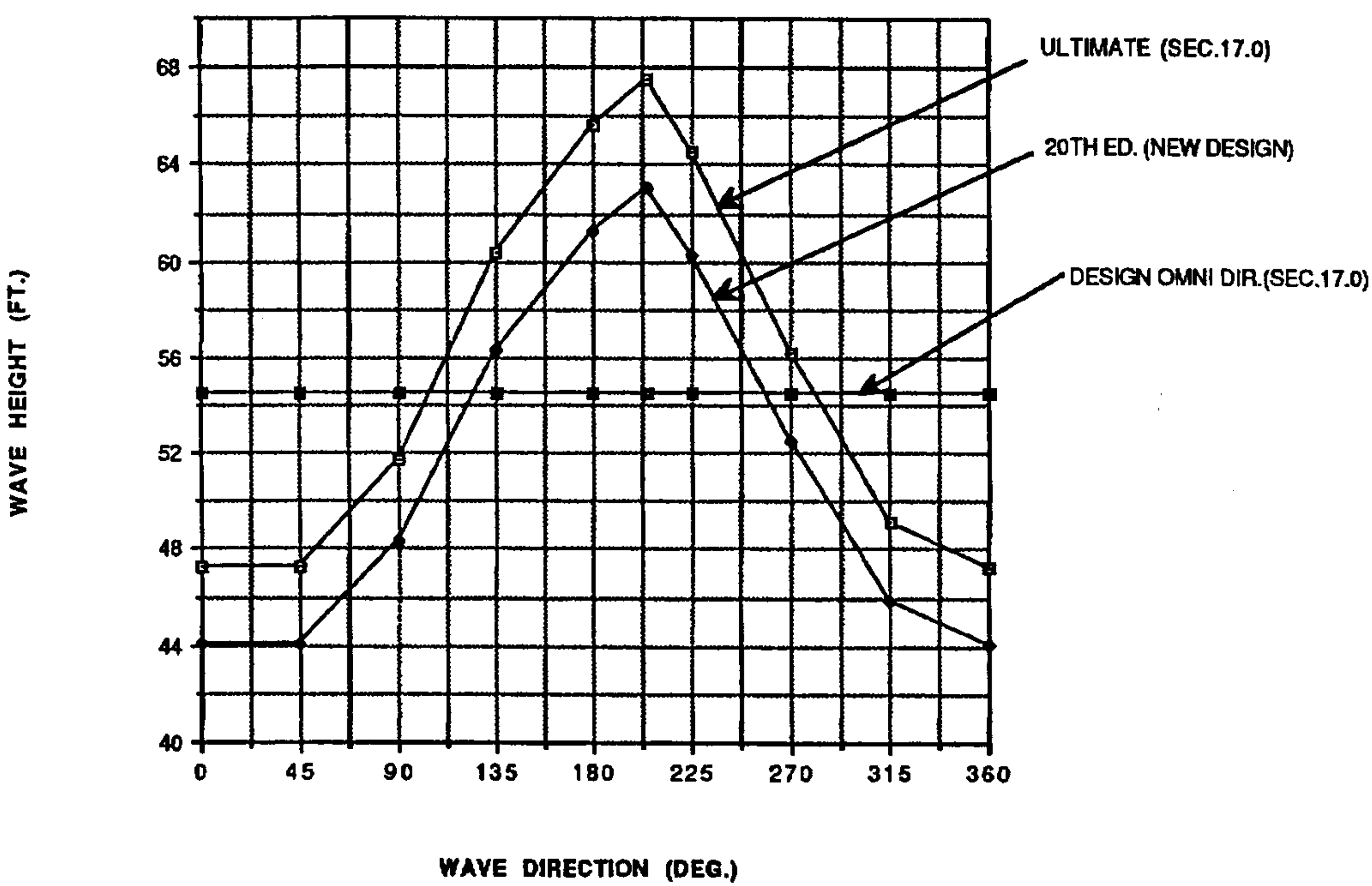


FIG. 3

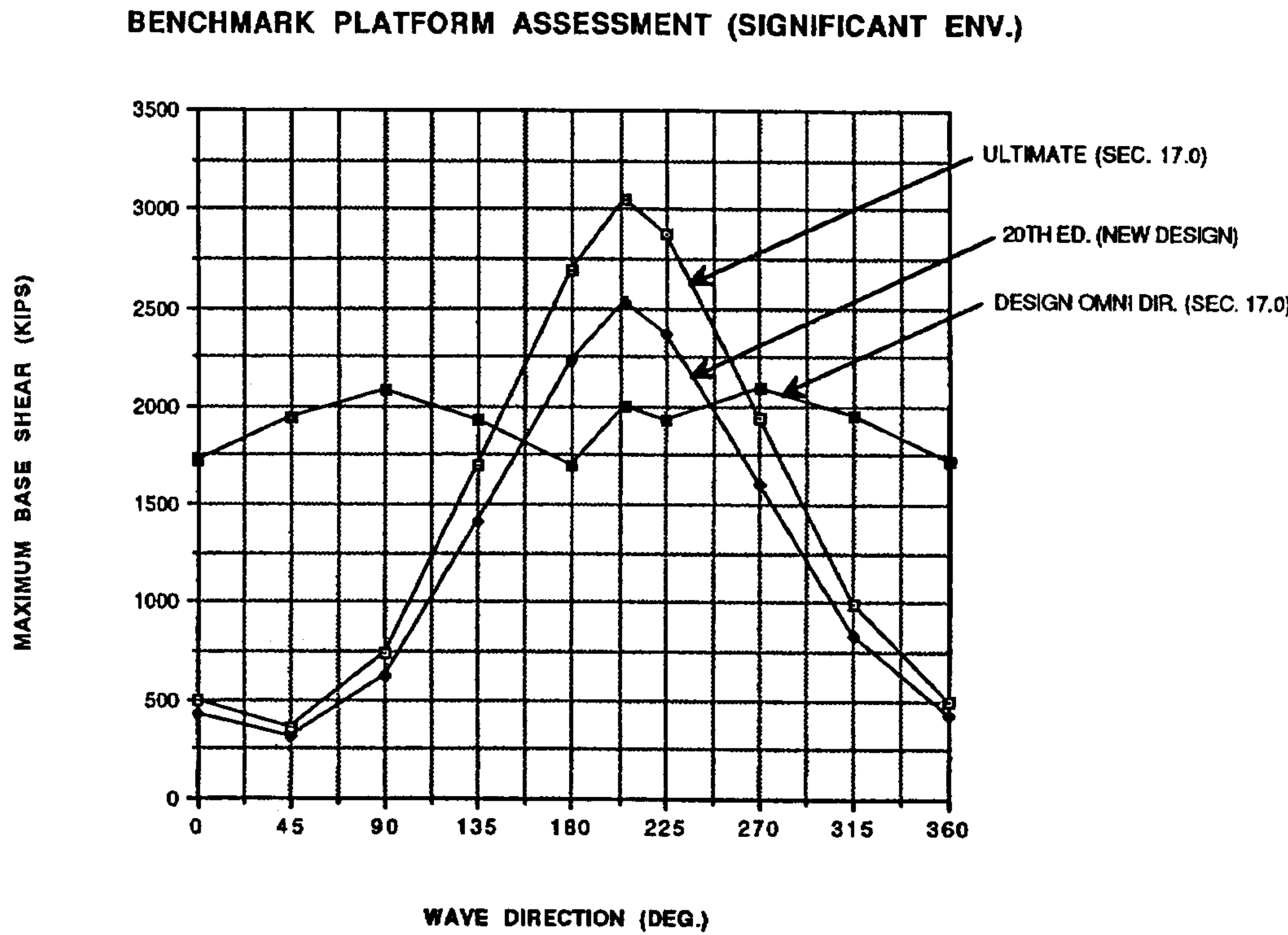

BENCHMARK PLATFORM ASSESSMENT (SIGNIFICANT ENV.)
MAXIMUM BASE SHEAR (KIPS)
3500
3000
2500
2000
1500
1000
500
0
0
45
90
135
180
225
270
315
360
WAVE DIRECTION (DEG.)
ULTIMATE (SEC. 17.0)
20TH ED. (NEW DESIGN)
DESIGN OMNI DIR. (SEC. 17.0)

FIG. 4

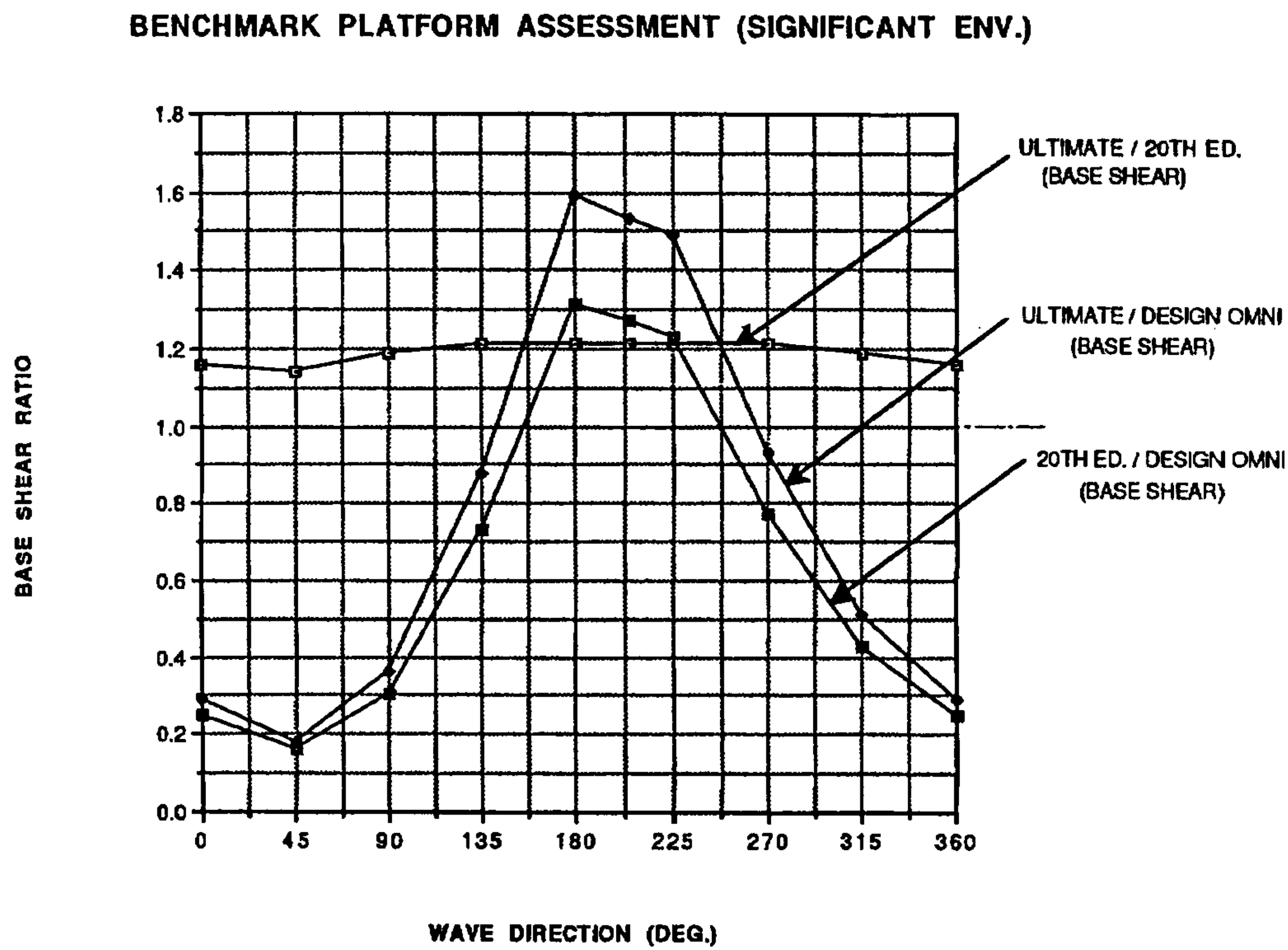

BENCHMARK PLATFORM ASSESSMENT (SIGNIFICANT ENV.)
BASE SHEAR RATIO
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
45
90
135
180
225
270
315
360
WAVE DIRECTION (DEG.)
ULTIMATE / 20TH ED. (BASE SHEAR)
ULTIMATE / DESIGN OMNI (BASE SHEAR)
20TH ED. / DESIGN OMNI (BASE SHEAR)

FIG. 5

BENCHMARK PLATFORM ASSESSMENT (SIGNIFICANT ENV.)

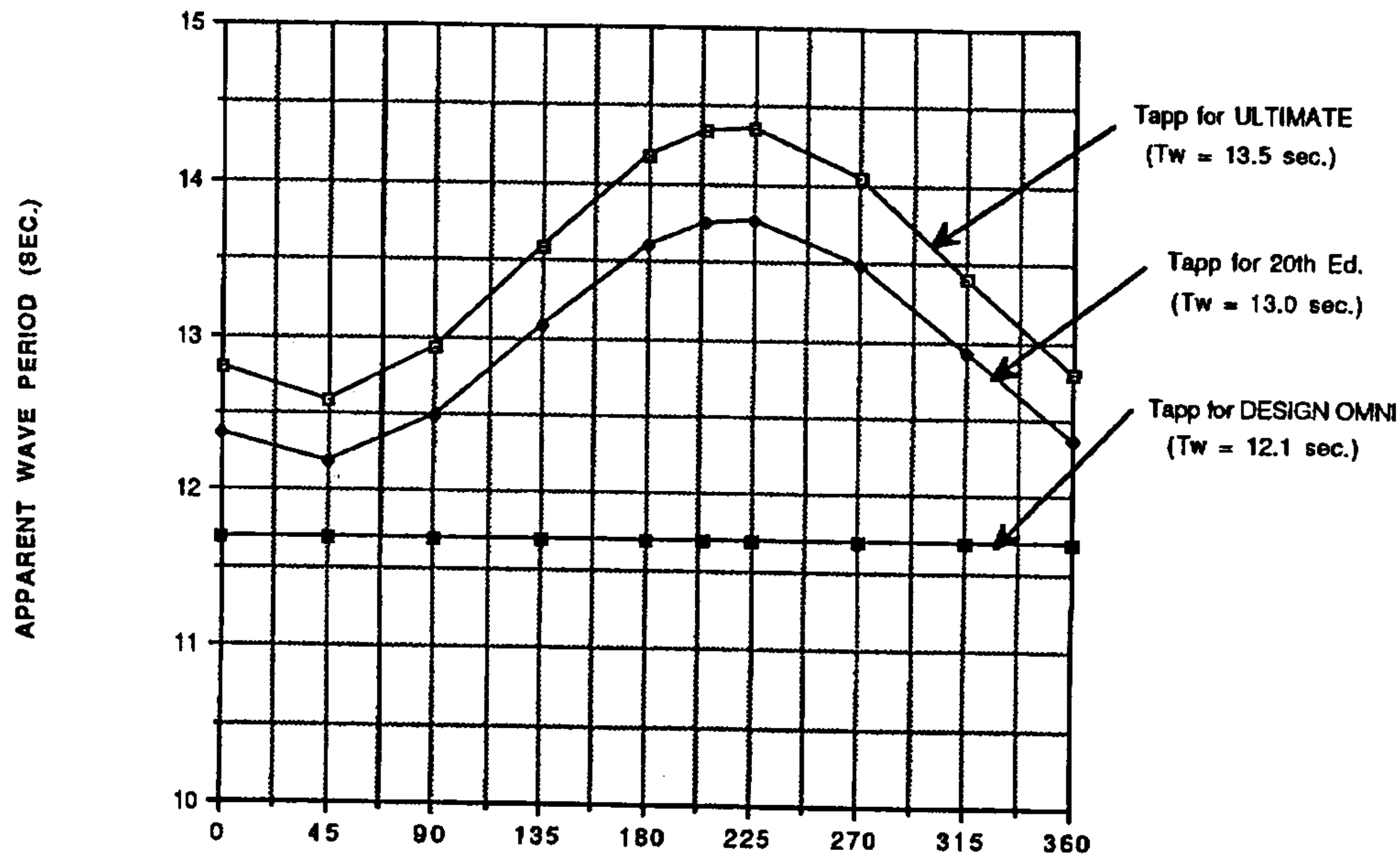


FIG. 6

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 205)

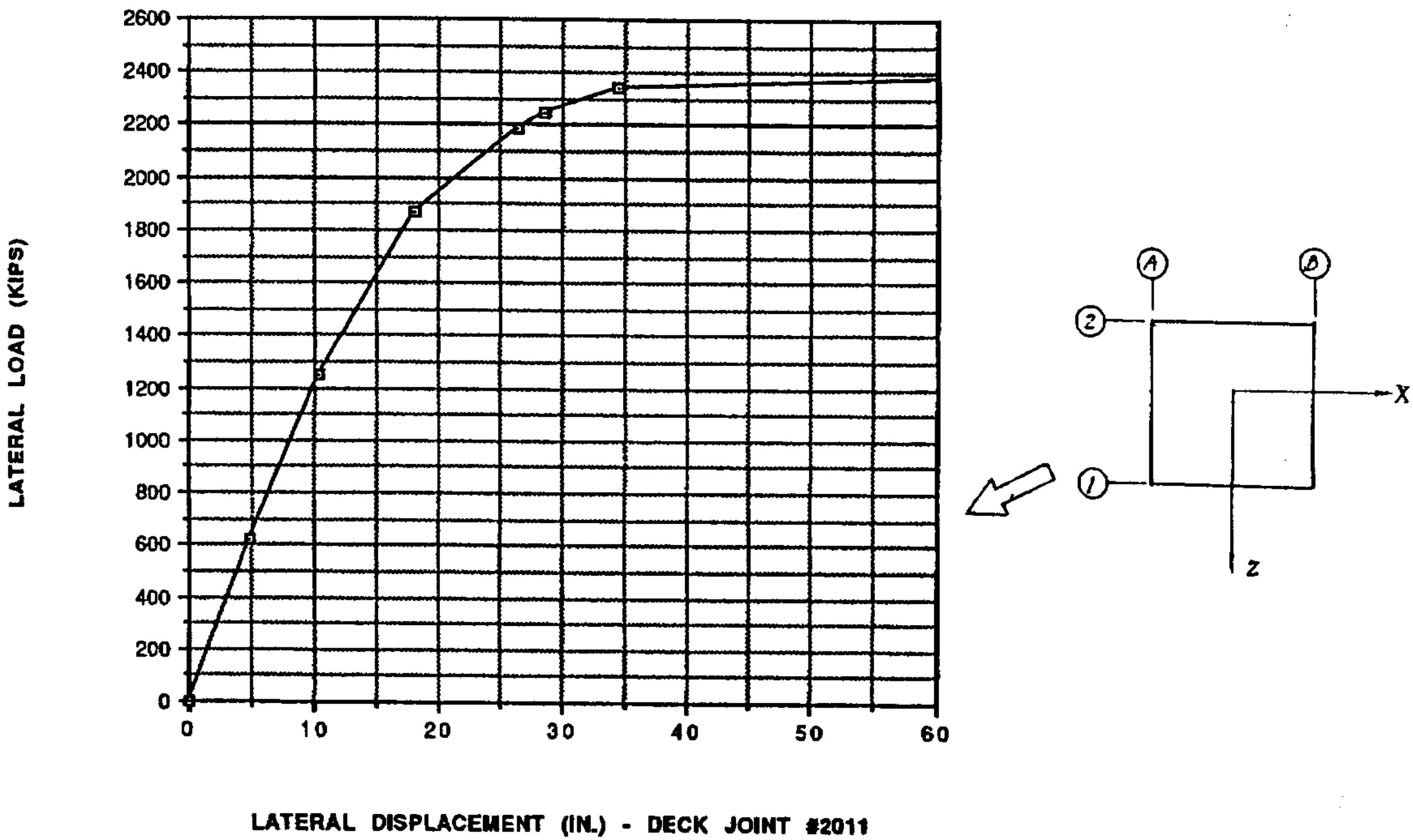


FIG. 7

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 205)

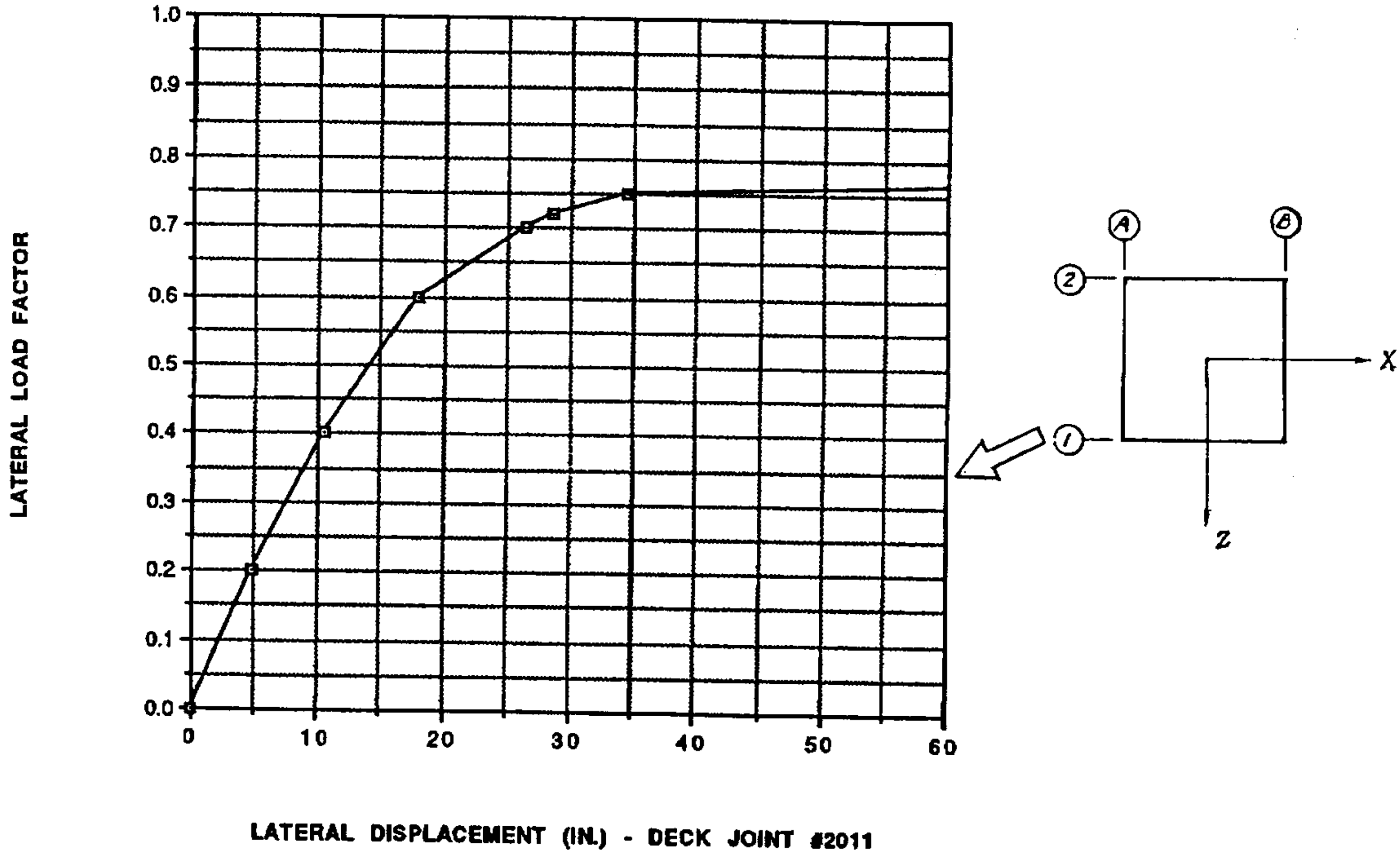


FIG. 8

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 180)

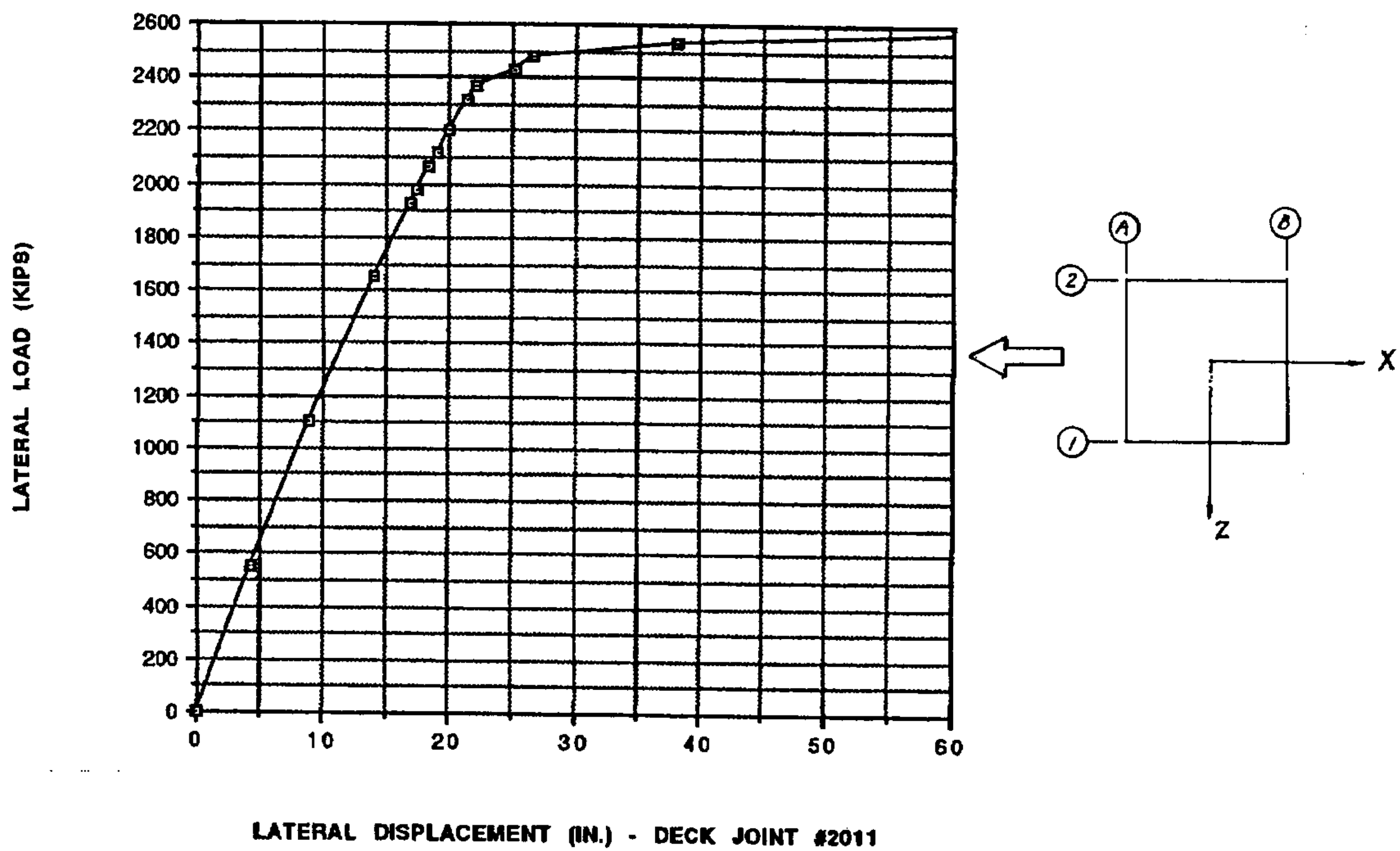


FIG. 9

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 180)

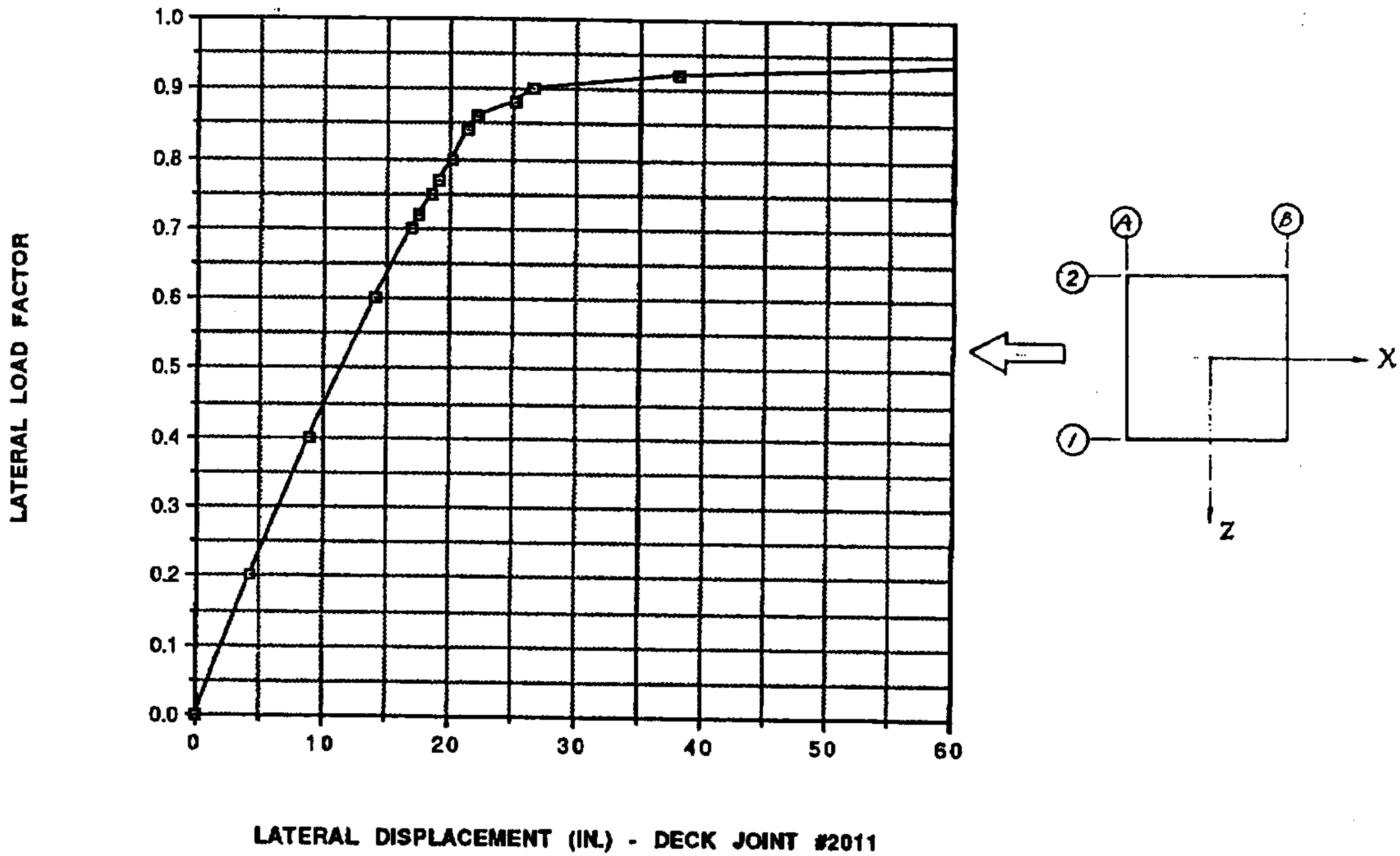


FIG. 10

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 225)

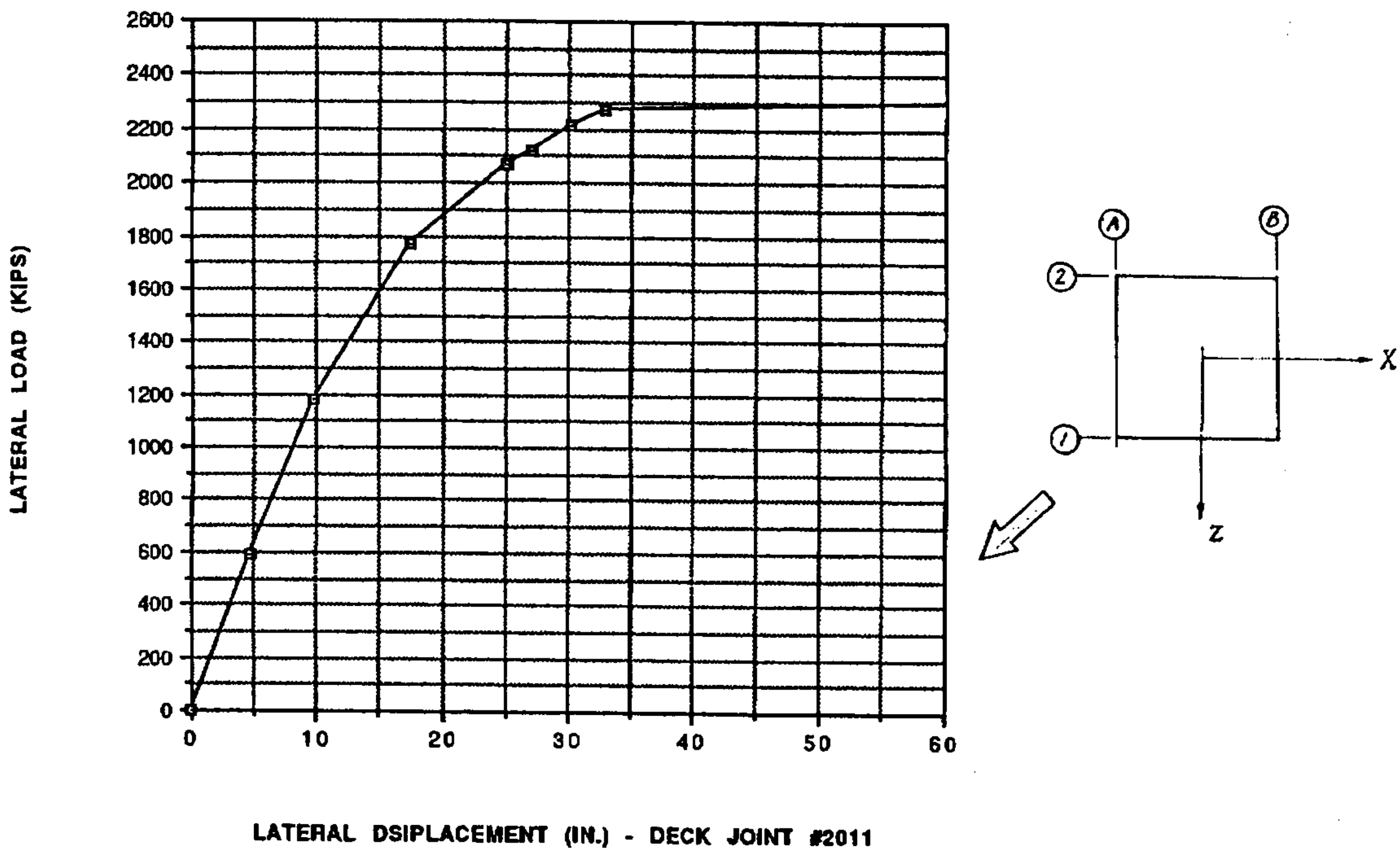
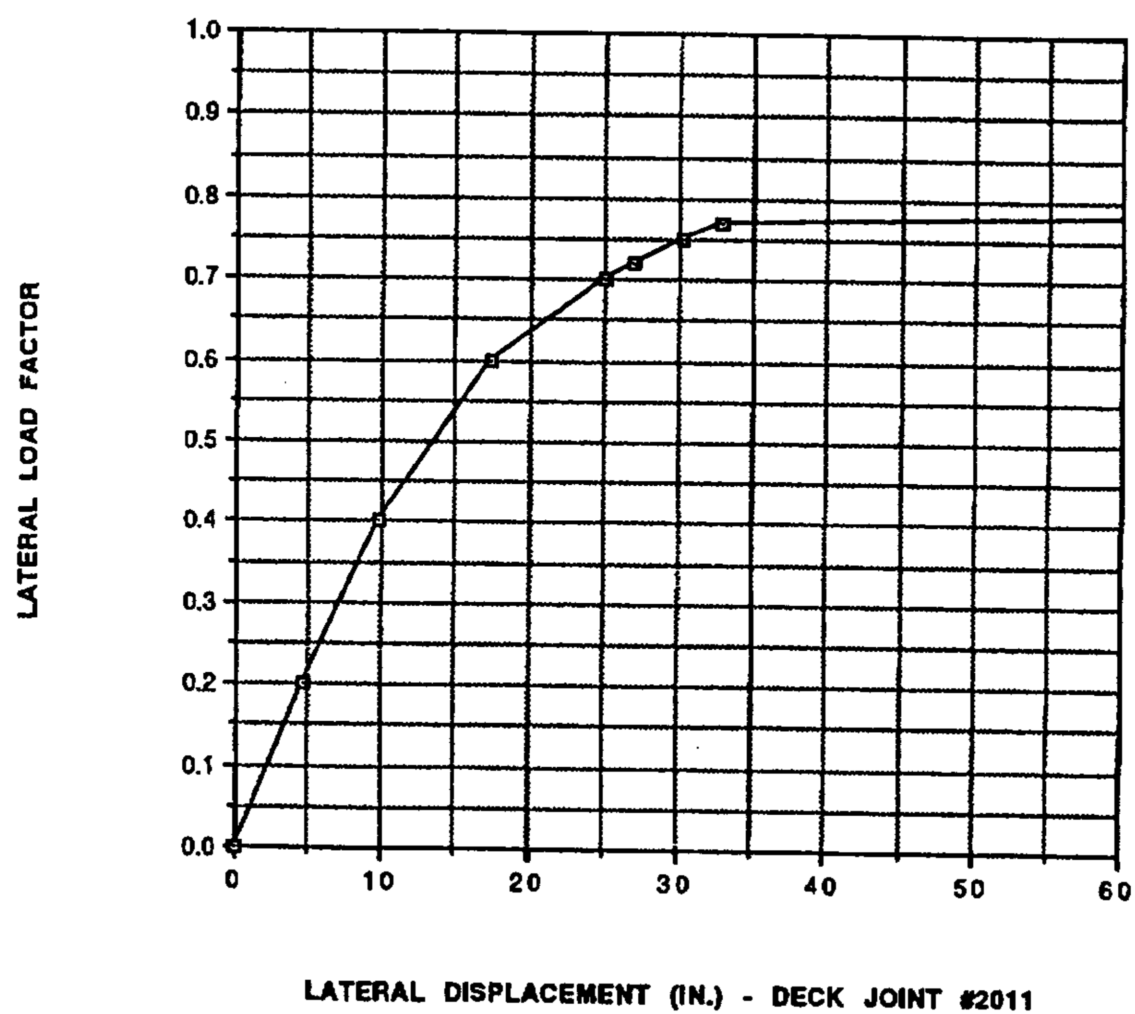


FIG. 11

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 225)



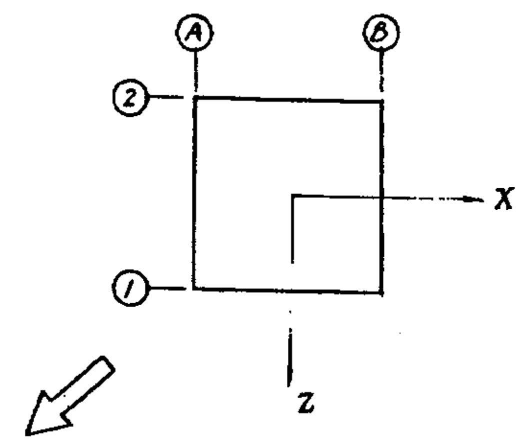


FIG. 12

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 270)

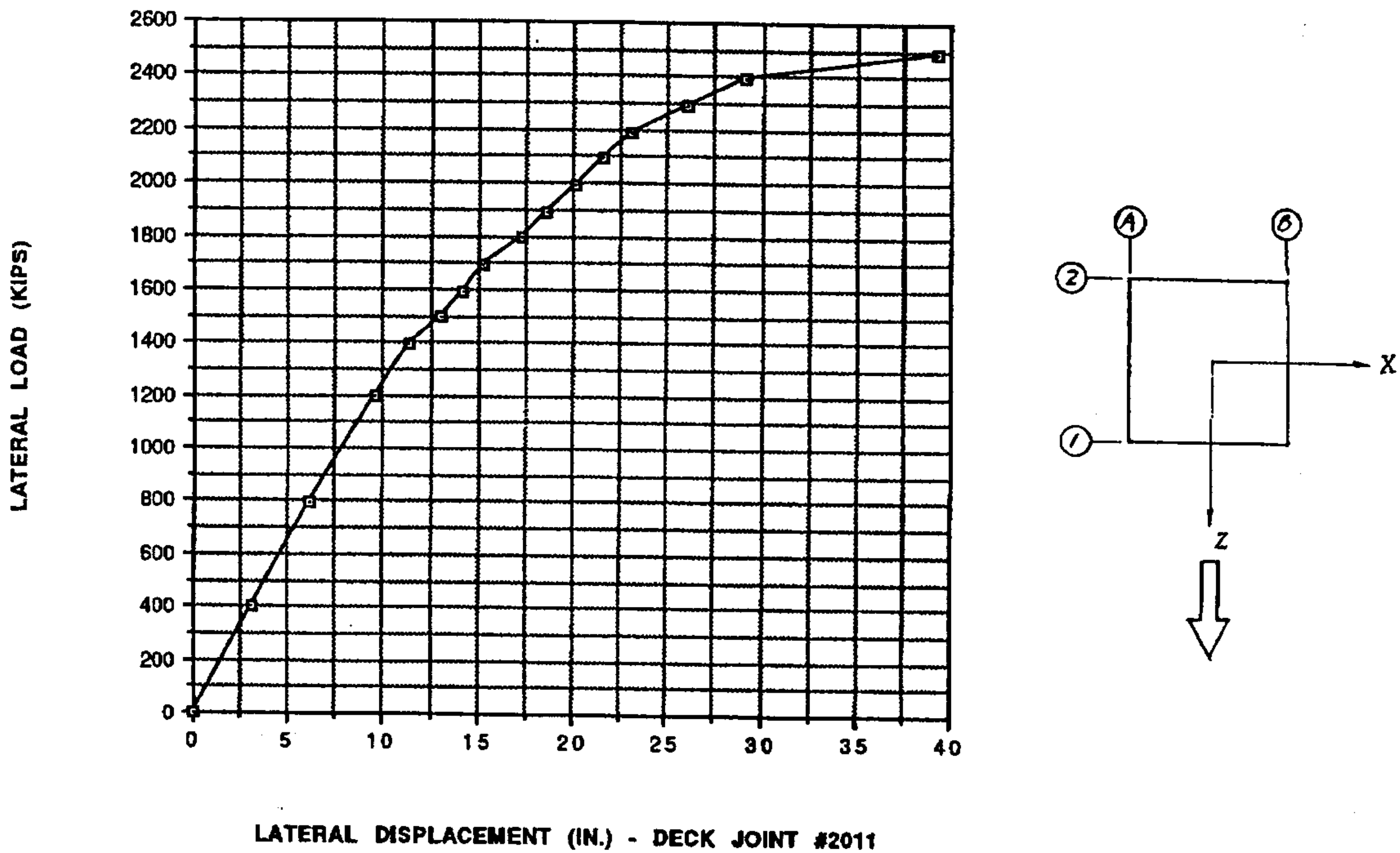


FIG. 13

BENCHMARK PLATFORM - STATIC PUSH-OVER ANALYSIS (WAVE DIR. 270)

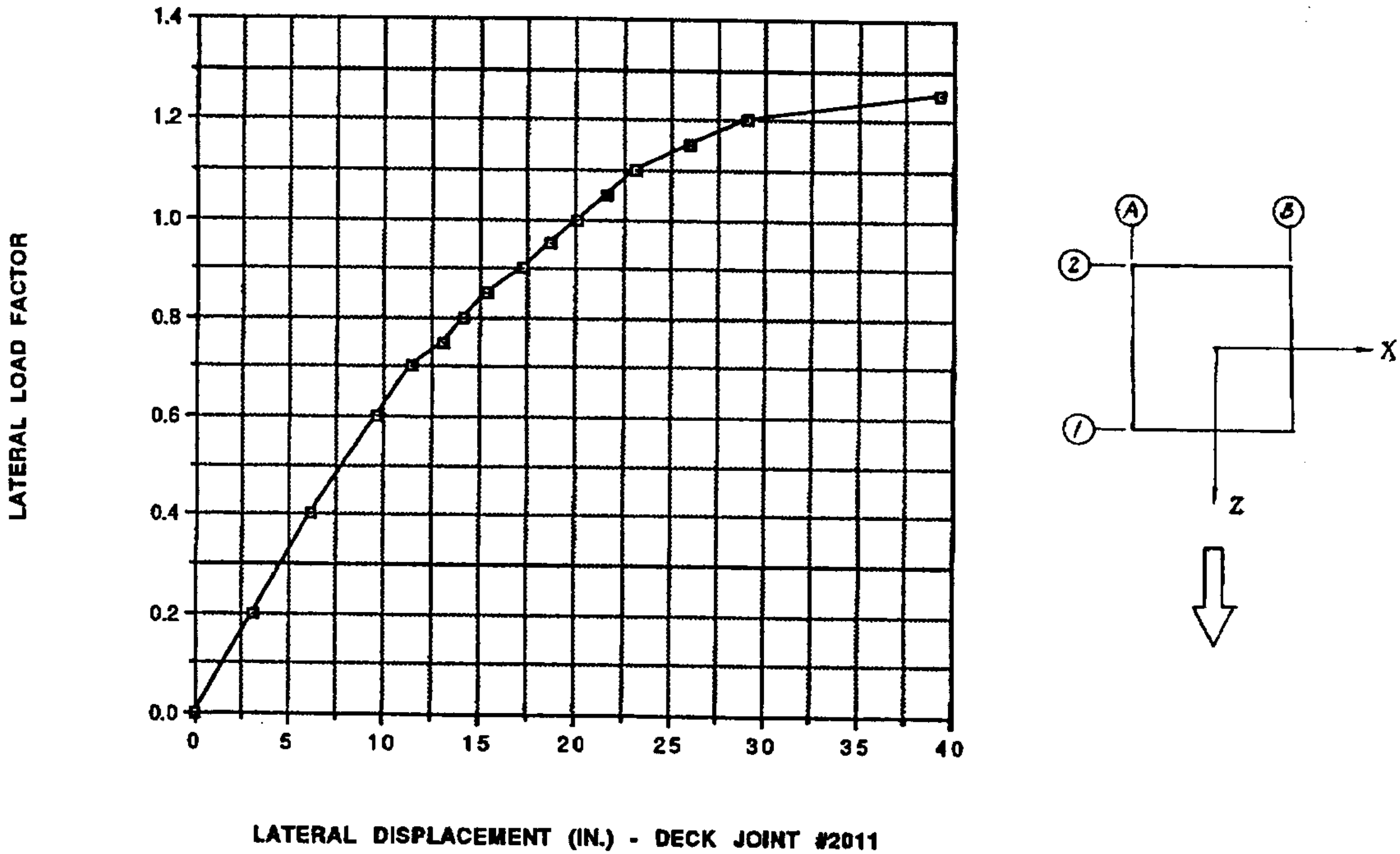


FIG. 14

APPENDIX

SOIL DATA FOR BENCHMARK (SHIP SHOAL BLK 220) Provided by 8/25/94

Pile Diameter 36"
Pile Penetration 355'

Pen = 0.0 ft		Pen = 5.0 ft		Pen = 10.0 ft	
y (in)	p (lbs/in)	y (in)	p (lbs/in)	y (in)	p (lbs/in)
0.18	13.73	0.18	38.53	0.18	66.15
0.54	19.81	0.54	55.58	0.54	95.41
0.90	23.48	0.90	65.89	0.90	113.12
1.26	26.27	1.26	73.71	1.26	126.55
1.80	29.59	1.80	83.02	1.80	142.52
3.60	37.28	3.60	104.60	3.60	179.57
5.40	42.61	5.40	119.55	5.40	205.23
27.00	0.00	27.00	39.85	27.00	136.82
54.00	0.00	54.00	39.86	54.00	136.82

Pen = 15.0 ft		Pen = 20.0 ft		Pen = 50.0 ft	
y (in)	p (lbs/in)	y (in)	p (lbs/in)	y (in)	p (lbs/in)
0.18	86.85	0.18	102.06	0.18	193.36
0.54	125.26	0.54	147.20	0.54	278.87
0.90	148.51	0.90	174.53	0.90	330.64
1.26	166.13	1.26	195.24	1.26	369.89
1.80	187.11	1.80	219.89	1.80	416.58
3.60	235.74	3.60	277.04	3.60	524.86
5.40	269.44	5.40	316.64	5.40	599.88
27.00	269.44	27.00	316.64	27.00	599.88
54.00	269.44	54.00	316.64	54.00	599.88

Pen = 72.2 ft		Pen = 72.3 ft		Pen = 100.0 ft	
y (in)	p (lbs/in)	y (in)	p (lbs/in)	y (in)	p (lbs/in)
0.18	260.92	0.09	261.22	0.09	345.52
0.54	376.31	0.27	376.75	0.27	498.33
0.90	446.17	0.45	446.69	0.45	590.83
1.26	499.12	0.63	499.70	0.63	660.96
1.80	562.14	0.90	562.79	0.90	744.40
3.60	708.25	1.80	709.07	1.80	937.89
5.40	809.47	2.70	810.42	2.70	1071.94
27.00	809.47	13.50	810.42	13.50	1071.94
54.00	809.47	27.00	810.42	27.00	1071.94

Pen = 158.0 ft		Pen = 158.1 ft		Pen = 197.0 ft	
y (in)	p (lbs/in)	y (in)	p (lbs/in)	y (in)	p (lbs/in)
0.09	522.03	0.06	522.33	0.06	640.71
0.27	752.89	0.19	753.33	0.19	924.07
0.45	892.66	0.32	893.18	0.32	1095.60
0.63	998.60	0.44	999.19	0.44	1225.64
0.90	1124.68	0.63	1125.33	0.63	1380.38
1.80	1417.00	1.26	1417.83	1.26	1739.16
2.70	1619.53	1.89	1620.48	1.89	1987.74
13.50	1619.53	9.45	1620.48	9.45	1987.74
27.00	1619.53	18.90	1620.48	18.90	1987.74

Pen = 197.1			Pen = 225.0			Pen = 225.1 ft	
y	p		y	p		y	p
(in)	(lbs/in)		(in)	(lbs/in)		(in)	(lbs/in)
0.50	33,401		0.50	38,129		0.06	735.75
1.00	51,699		1.00	59,017		0.19	1061.14
1.50	58,601		1.50	66,896		0.32	1258.11
2.00	60,825		2.00	69,435		0.44	1407.44
2.50	61,505		2.50	70,211		0.63	1585.13
3.00	61,709		3.00	70,444		1.26	1997.13
4.00	61,788		4.00	70,535		1.89	2282.58
5.00	61,795		5.00	70,543		9.45	2282.58
6.00	61,796		6.00	70,543		18.90	2282.58
42.00	61,796		42.00	70,543			
Pen = 346.0 ft							
y	p						
(in)	(lbs/in)						
0.06	735.75						
0.19	1061.14						
0.32	1258.11						
0.44	1407.44						
0.63	1585.13						
1.26	1997.13						
1.89	2282.58						
9.45	2282.58						
18.90	2282.58						

SOIL DATA FOR BENCHMARK (SHIP SHOAL BLK 220) Provided by Chuck Kindel						8/18/94	
Pile Diameter 36"							
Pile Penetration 355'							
Pen = 0.0 ft			Pen = 5.0 ft			Pen = 10.0 ft	
Z	T		Z	T		Z	T
(in)	(lbs/in2)		(in)	(lbs/in2)		(in)	(lbs/in2)
0.058	0.164		0.058	0.225		0.058	0.286
0.112	0.274		0.112	0.375		0.112	0.476
0.205	0.411		0.205	0.563		0.205	0.714
0.288	0.493		0.288	0.675		0.288	0.857
0.360	0.548		0.360	0.750		0.360	0.953
0.720	0.493		0.720	0.675		0.720	0.857
36.000	0.493		36.000	0.675		36.000	0.857
Pen = 15.0 ft			Pen = 20.0 ft			Pen = 50.0 ft	
Z	T		Z	T		Z	T
(in)	(lbs/in2)		(in)	(lbs/in2)		(in)	(lbs/in2)
0.058	0.346		0.058	0.407		0.058	0.771
0.112	0.577		0.112	0.679		0.112	1.286
0.205	0.866		0.205	1.018		0.205	1.929
0.288	1.039		0.288	1.222		0.288	2.314
0.360	1.155		0.360	1.357		0.360	2.571
0.720	1.039		0.720	1.222		0.720	2.314
36.000	1.039		36.000	1.222		36.000	2.314
Pen = 100.0 ft			Pen =197 ft			Pen = 197.1	
Z	T		Z	T		Z	T
(in)	(lbs/in2)		(in)	(lbs/in2)		(in)	(lbs/in2)
0.058	1.379		0.058	2.556		0	0.000
0.112	2.298		0.112	4.260		3.6	11.806
0.205	3.446		0.205	6.391		36	11.806
0.288	4.136		0.288	7.669			
0.360	4.595		0.360	8.521			
0.720	4.136		0.720	7.669			
36.000	4.136		36.000	7.669			
Pen = 225.0 ft			Pen = 225.1 ft			Pen = 346.0 ft	
Z	T		Z	T		Z	T
(in)	(lbs/in2)		(in)	(lbs/in2)		(in)	(lbs/in2)
0	0.000		0.058	2.935		0.058	2.935
3.6	11.806		0.112	4.892		0.112	4.892
36	11.806		0.205	7.339		0.205	7.339
			0.288	8.806		0.288	8.806
			0.360	9.785		0.360	9.785
			0.720	8.806		0.720	8.806
			36.000	8.806		36.000	8.806

						End Bearing Curve	
Pen = 355.0 ft						Pen = 355.0 ft	
Z	T					Z	Q
(in)	(lbs/in2)					(in)	(lbs)
0.058	3.378					0.072	25853.405
0.112	5.630					0.468	51706.809
0.205	8.445					1.512	77560.214
0.288	10.133					2.628	93072.256
0.360	11.259					3.600	103413.618
0.720	10.133					36.000	103413.618
36.000	10.133						

PARTICIPANT "J"

1.0 Summary

The ultimate strength analysis of the Benchmark platform was carried out using the software system, RASOS - Reliability Analysis System for Offshore Structures. This software is described in greater detail in Section A.3.

The analysis model was generated as a 3-D frame structure with tubular beam members for the jacket structure and I-beam members for the platform deck.

The environmental load due to the combined action of wave current and wind loads was generated with marine growth and aero- and hydro-dynamic statistical modelling taken into account. Gravity effects were also included.

Linear elastic analysis was carried out on the frame. User defined permanent load sets were specified to account for the different deck loads. Response from the generated environmental loads and gravity loading was also calculated. The axial capacity and initial softening slope in compression were calculated using a non-linear beam-column element based on a 3-D elasto-plastic buckling analysis.

An algorithm using a non-linear beam/column element based on a 3-D elasto-plastic buckling analysis was employed to represent the member post limit behaviour.

The collapse analysis was based on the Virtual Distortion Method which involves scaling up the initial vector of member forces, until the structure became a mechanism. Once a member failed, either by buckling, plasticity or fracture, the load supported by this member was determined by its post limit behaviour. The surplus force was re-distributed to the other members by means of virtual distortions. This procedure was repeated for every failed member during the loading process.

The Benchmark platform was analyzed in three different directions of environmental loading that would model worse case scenarios. These were in North, North-East and East approach directions with respect to the Platform North. The platform was modelled as either being fully fixed at base or having pile springs at the feet of the platform legs. The displaced shapes at ultimate load varied considerably depending upon whether pile spring or fully fixed models were present. The structure had adequate reserve strength ratios to survive the 100 year return period load in the North and East environment load approach directions, but had low redundancy values in these directions. The structure under the North East environmental loading had insufficient reserve strength ratio with a fully fixed base, but sufficient reserve strength ratio with a pile restrained base. In both cases it had much larger redundancy under this loading. Chord and brace members in the region between 130 ft and 157 ft elevation above the mud line were predicted to fail first when using the pile spring model; whilst chord members at the feet of the structure's legs were predicted to fail first when using fully fixed model.

A.1 Environmental Criteria

For the ultimate strength analysis, three different wave approach directions were used: These were 270°, 180° and 225° with respect to the platform North. These three directions give an East / West face direction, a North / South face direction and a North East / South West diagonal direction in the principal quadrant of the wave directions given in figure 2.3.4-4 of API.

The following table 1, shows the sea state parameters used in the analysis for each wave approach direction. The wave height was taken from Fig 17.6.2-2a of API RP2A-WSD SEC17, for a "Full Population Hurricane" with an ultimate strength analysis, giving a base omni-directional value of 67ft. This value was then multiplied by the appropriate wave reduction factor interpolated from the values given in Figure 2.3.4-4 of API 20th Ed. for each of the three directions analyzed.

The wave period was taken from Table 17.6.2-1 of API RP2A-WSD SEC17, giving a value of 13.5 secs. Fig 2.3.1-2 of API 20th Ed. was then used to calculated the doppler shift due to steady current on the wave period, and hence the apparent wave period, giving the value shown below. The wind speed was also taken from the above mentioned table.

The current speed was taken from Table 17.6.2-1 of API RP2A-WSD SEC17, as 2.3 kts, the current direction was taken from Fig. 2.3.4-5 of API 20th Ed., and was found to be 283°, the equivalent current speed in each of the directions analyzed was found by resolving this vector in each direction. From Fig. 2.3.4-6, it was found that a uniform current profile should be used. Current blockage factors were taken from Section 2.3.1b.4 to take account of the reduction in current speed around the jacket legs, these values are shown below for the three directions analyzed.

The storm surge was found to be 4ft from fig.17.6.2-2a, this was added to the water depth giving a total water depth of 161ft used in the analysis:

Section 2.3.4d.1 specifies a wave kinematics factor of 0.88. and according to Morrisons equation, given in Section 2.3.1b.10, this is applied to the water particle velocities and accelerations due to the wave. Since the marine growth is taken to be 1.5 inches (Section 2.3.4d.2) the jacket structure is assumed to be rough and hence the hydrodynamic base drag coefficient is taken as 1.05 and the hydrodynamic inertia coefficient as 1.2. For the aerodynamic coefficients the base drag coefficient is taken as 0.65 and inertia coefficient as 1.6

From Fig 2.3.1-3 it was found that the appropriate wave theory for this analysis was the Stream Function of the 7th order. However, RASOS is currently limited to using the Airy's linear wave theory.

Wave Approach Direction (w.r.t. Platform North)	East / West	North / South	North East / South West
Wave height (ft)	65.124	55.811	64.022
Kinematics factor	0.88	0.88	0.88
Apparent Wave period (sec)	14.5	14.5	14.5
Current speed (ft/sec)	3.296	2.059	3.786
Current Blockage factor	0.80	0.80	0.85
Storm surge (ft)	4.0	4.0	4.0
Wind speed (1 hr @ 10 m) (ft/sec)	143.62	143.62	143.62

Table 1: Sea state parameters for three wave approach directions

The wind load, calculated from the projected deck areas provided by PMB, using equations given in 2.3.2b and 2.3.2c of API 20th Ed., was resolved to two equal resultant forces and moments applied as nodal forces on to the top nodes of the side of the structure facing the loading direction. The values and directions of these loads are shown in table 2 below.

Wave Approach Direction (w.r.t. Platform North)		East / West		North / South		North East / South West	
Node Number		1105	1107	1101	1107	1101	1105
Force	Direction	-x	-x	y	y	-x=y	-x=y
	Value kN	358.5	358.5	316.2	316.2	255.8	255.8
Moment	Direction	$-R_y$	$-R_y$	R_x	R_x	$R_x=-R_y$	$R_x=-R_y$
	Value kNm	2836.4	2836.4	2358.4	2358.4	1973.3	1973.3

Table 2: Wind loads applied to the platform deck projected areas

The positioning for the four conductors (C_1, C_2, C_3, C_4) within the jacket structure meant that depending on the wave approach direction, certain shielding effects had to be taken into account with respect to the drag and inertia coefficients used in the analysis. The shielding factors were applied to these coefficients for the members representing the affected conductors in each wave approach direction. From figure 2.3.1-4 of API, the following shielding factors were calculated for the three wave approach directions.

Wave Approach Direction (w.r.t. Platform North)	East / West	North / South	North East / South West
C_1 shielded by C_2	0.45	-	-
C_1 shielded by C_3	-	-	0.45
C_2 shielded by C_1	-	0.55	0.45
C_2 shielded by C_3	0.7	-	-
C_3 shielded by C_4	0.45	0.55	0.71

Table 3: Shielding factors applied to the drag and inertia coefficients for the four conductors

There is no facility within the RASOS software system to evaluate the stream function (see fig. 2.3.1-3 of API) and hence the wave/current platform deck forces. This, therefore had to be omitted from the analysis. The implication of this on the computed results is discussed in section A.4.

A.2 3-D Model Generation

The computer model was approximated as follows. Tubular elements were used to model the jacket structure, while I-beams elements were used for the platform deck elements. The deck was only modelled as far as the second deck level, 43' above sea level. The base of the jacket structure was modelled as far as the mud line with soil springs at the base of the legs to simulate the pile stiffness. Soil springs were also used at the base of the conductors to model the effects of the length of the conductors below the mud line. The conductors were constrained to the horizontal levels within the jacket structure by the use of constraint equations. The pile members were also constrained but to the jacket legs in the horizontal directions at the different levels of the jacket structure. The risers were modelled as secondary members which transfer only the horizontal forces but no moments to the jacket, also their contribution to the stiffness of the jacket structure is ignored. Figures 1 & 2 show the F.E. plots of the benchmark platform model from a North-South view and East-West view, respectively.

The elements selected to be considered as non-linear (ie, potentially damaged) were: the horizontal jacket members, the primary bracing in the horizontal and vertical faces, the primary deck bracing, the jacket legs and the piles.

The pile analysis program SPLINTER, part of the ASAS suite of programs (ref: "SPLINTER User Manual", ASAS Finite Element System, Version H11, WS Atkins Engineering Software, 1993) was used to evaluate spring stiffnesses in six degrees of freedom at the pile cap. The SPLINTER program is based on the subgrade reaction model that applies horizontal and vertical load-displacement soil models along the length of the pile.

The lateral load-displacement soil model used in SPLINTER is based on the 15th API (1984) recommendations. The skin friction and end bearing soil models are based on Vijayvergia's (1977) recommendations. The stiffness of the soil is based on the SECANT stiffness technique.

The pile was modelled as a series of tubular elements between the mud line and the pile tip, 355 ft below the mud line with a 7.34° rake angle from the vertical. The dimensions of the tubular elements reflected the actual external dimensions of the pile as seen by the soil. The wall thickness of the pile varied from one tubular element to the next. The top 12 ft of the pile was modelled with a wider external diameter and larger wall thickness to account for the effect of the jacket sleeve. The four piles of the jacket were sufficiently far apart at the mud line level to allow the piles to be analyzed individually without reference to the effects of the other piles.

The pile was considered to be plugged. Two different types of clay were modelled from 0.0 ft to 197 ft below the mud line and from 225 ft to 391 ft below the mud line. A single type of sand was modelled from 197 ft to 225 ft below the mud line.

From the results of the analysis of motion versus reaction force in a single degree of freedom, a spring stiffness was evaluated. When the base reactions and over-turning moments were known from the results of a preliminary linear analysis with fixed base, the appropriate fully populated stiffness matrix could be generated.

Within the global analysis, a selection is made to choose the most critical combination of various limit state models from material, beam/column and joint strength. These values are then used to construct the final limit surface for the collapse analysis.

A.3 Software Description

The Software system, RASOS - Reliability Analysis System for Offshore Structures is an advanced computer code designed for structural system reliability analysis.

The main functions of RASOS are concerned with:

i. Modelling the stochastic nature of the environmental variables and resistance parameters.

ii. Modelling the physical behaviour of the structure.

iii. Evaluation of the reliability of individual components and failure modes.

iv. Evaluation of the system reliability and interpretation of the results.

Within RASOS the physical behaviour of the structure under extreme loading is analyzed using the Virtual Distortion Method - VDM. This is a highly efficient method of progressive collapse analysis and is particularly attractive for the repetitive analysis which is essential for reliability evaluation. The program is applicable to the quasi-static analysis of skeletal structures. The behaviour of failed elements is analyzed using a sophisticated, non-linear, beam-column model. This produces behaviour which is then used by VDM to model the global structural response.

RASOS is a suite of eleven closely interlinked software modules for the reliability analysis of skeletal offshore structures. It offers a number of physical sub-models and a range of alternatives for both progressive collapse analysis and structural reliability analysis. The five main modules used in this analysis are described below.

RASOS_M Model generation for frame and truss structures by input of structural geometry and material data. Facilities include the modelling of 2-D or 3-D frame structures with tubular beam or general beam members, joint identification, selection of flexible joints, can and stub data and generation of the appropriate tables.

RASOS_E Generation of environmental load due to the combined action of the eight deterministic components of environmental loading: wave current and wind loads, with marine growth and aero- and hydro-dynamic statistical modelling taken into account. A response surface approach based on Airy wave theory is used to compute components of the total loading. Both distributed member forces and mean value of the equivalent nodal vector forces acting on the structure are computed. An algorithm giving the random multipliers of the response surface allows a large number of simulations to be carried out efficiently. Gravity and buoyancy effects can be included.

RASOS_L Linear elastic analysis of frames with rigid or flexible joints and generation of the structural stiffness matrix. Facilities include the analysis of multiple load sets, user defined permanent or variable load, response from generated environmental load - nodal forces and uniformly distributed user defined loading, gravity and thermal loading. Calculation of the member internal forces and the nodal displacements, separately for the eight deterministic environmental load sets and the combined response for environmental, gravity and user defined load sets. An algorithm incorporated in RASOS is used to determine the post limit member behaviour. The axial capacity and initial softening slope in compression are calculated using a non-linear beam-column element based on a 3-D elasto-plastic buckling analysis. The results from this analysis for each structural member are passed on to the global collapse analysis so as to obtain a description of each member's post limit behaviour.

RASOS_D Generation of the structural influence matrix. Facilities include definition of non-linear members, generation of non-linear member properties, definition of limit surface, calculation of member capacities and member - structure interaction springs, to be used during deterministic collapse analysis or system reliability analysis.

RASOS_C Deterministic progressive collapse analysis based on the Virtual Distortion Method (VDM). This concept uses the superposition principle where two states of the structure are combined - a fundamental state due to the original linear-elastic solution and a virtual state caused by virtual distortions introduced into the structure to account for the non-linear behaviour. Member strength is calculated based on a non-linear beam-column model. Various local strength models are accounted for including simplified elasto-plasticity with hardening/softening, brittle or ductile fracture, compressive strength reduction due to member buckling and joint strength. Intact or damaged members can be considered, including members with initial imperfections. Global boundary conditions allow for non-linear support springs. Computational strategy includes non-proportional, incremental loading and unloading.

A.4 Ultimate Strength Analysis

The ultimate strength analysis was carried out in the three wave approach directions for two support conditions: Firstly, when the jacket legs are fully fixed at the mud line, and secondly when the soil springs are modelled at the mud line.

The member capacities are determined using a member-in-isolation analysis, with the non-linear beam/column model. In order to carry out such an analysis, it is necessary to represent the stiffness of the rest of the structure via end springs attached to the member. The stiffnesses of these springs are calculated by using VDM. The member - in - isolation analysis also requires member loads to be applied incrementally as a result of the non-linearities involved. This is carried out by assuming that the same factors are applied to various load components as in the original linear analysis. Thus only the independent load controlling parameter is required. Because of the nature of the numerical algorithm developed in the beam/column model, the axial load on a member is selected to be the controlling factor. The effective length factors (K-factors) are derived in RASOS from the member-structure interaction springs and not from the API codes. **These springs represent the restraint provided by the rest of the structure on the member under consideration.** In the API code, empirical equations for the member capacities are given but no information is available about its post-limit behaviour. In RASOS, however, post limit behaviour is accounted for.

The limit state model for joints is based on the formulae published in the Underwater Engineering Group (UEG) report, "Design of tubular joints for offshore structures", UEG Publications UR33, 1985. In this report, the expressions for mean and characteristic ultimate axial load, in-plane moments and out-of-plane moment for joints were derived from a large data base of experimental results on T/Y, K and X joints. The mean limit state formulae have been adopted for the joints strength model in RASOS. It is also assumed that the **joints are considered simple (ie no overlapping) and that the post limit behaviour of the joints is fully plastic.** In the **API code, joint capacities are based on the loads causing first cracking, however, in RASOS, the ultimate strength of the joints is used.**

RASOS allows the environment loading and non-linear properties to be calculated in separate modules to the collapse analysis, allowing many collapse analyses to be performed within a short space of time.

The collapse analysis is carried out by scaling up the initial vector of member forces, until the structure becomes a mechanism. Once a member fails, either by buckling, plasticity or fracture, the load supported by this member is determined by its post limit behaviour. The surplus force is re-distributed to the other members using the Virtual Distortion Method, a concept using the superposition principle where two states of the structure are combined - a fundamental state due to the original linear-elastic solution and a virtual state caused by virtual distortions introduced into the structure to account for the non-linear behaviour. This procedure is repeated for every failed member during the loading process.

The load incrementing used during the collapse analysis is a strain controlled process at element level. This allows accurate tracing of the non-linear behaviour of the structure. The ultimate load and the sequence of member failures with associated load levels and global damage indicators are given.

The loads were applied incrementally to the structure such that the deck (live and dead) loads, the wind loads exerted on the deck and the gravity loading were applied in the first load increment, with the environmental load applied gradually in subsequent increments.

Presentation and Discussion of Results

The lateral load (P) - deflection (Δ) plots at the South East leg elevation at +45 ft above sea level are shown in:

- Figures 5 & 8 for the East-West environmental loading approach with fully fixed and pile restrained models, respectively;
- Figures 11 & 14 for North-South environmental loading approach with fully fixed and pile restrained models, respectively;
- Figures 17 & 20 for North East-South West environmental loading approach with fully fixed and pile restrained models, respectively.

From these figures it is observed that the jacket structure can survive the design load for all environmental load approaches and jacket models, except for the fully fixed model in the North East-South West (NE-SW) environmental loading approach. The jacket structure responds in a distinctly non-linear fashion to the NE-SW environmental loading approach, for both the fully fixed and pile restrained models. The load level at first member failure (S_1) is very close to the ultimate capacity (R_u), for both the East-West and North-South cases; whereas in the NE-SW case the first member failure is significantly smaller than the ultimate capacity. Tables 4, 6, 8, 10, 12 and 14 contain tabulated values of the P-Δ plots.

Figure 3 & 4 show F.E. plots of the benchmark platform with some specific member and node numbers where plastic hinges have formed. The displaced shape (m) at ultimate load of the fully fixed base and pile restrained structures are shown in Figures 6 and 9 for the East-West environmental loading approach. The magnification factor for the fully fixed case is 406 compared with 80.6 in the pile restrained case, which from the figures may be interpreted as the pile restrained structure deflecting more than the fully fixed structure at ultimate load. The platform East-West environmental loading is parallel to the y-axis in the negative direction, which corresponds to the direction of the observed displacements in the figures. Tables 5 and 7 record the member failure sequence for fully fixed and pile restrained models, respectively. The first member failure occurs at 1.17 times the design shear load for the fully fixed model, which is slightly less than that for the pile restrained case at 1.196.

Importantly, the members that fail are different i.e. member 9931 and 7181 for the fully fixed and pile restrained models respectively, see Figures 3 & 4. That is the fully fixed model starts to collapse at the foot of the North West leg, highlighted in red in Figure 7 which shows the utilization ratio (equivalent member force). Equivalent member force is the ratio of the member loads to its capacities, in axial load, inplane and out-of-plane bending. In the pile foundation case the first members to fail are the chord members in the North West and South West legs between elevation 130 ft and 157 ft above the mud line. From Figures 7 & 10 it is observed that the same diagonal braces on the Platform North side have very high utilization ratios, shown in red.

The displaced shape (m) at ultimate load of the fully fixed base and pile restrained structures are shown in Figures 12 and 15 for the North-South environmental loading approach. The magnification factor for the fully fixed case is 120 compared with 53.7 in the pile restrained case. Again it may be interpreted from the figures that the pile restrained structure deflects more than the fully fixed structure at ultimate load. The platform North-South loading is parallel to the y-axis in the negative direction. However, the displaced shapes appear to contradict one another i.e. the pile restrained model is displaced in the direction of the environmental loading whereas the fully fixed shape is displaced in the direction opposite to the environmental loading. In fact the displaced shape of the fully fixed model, which appears incorrect can be understood as a result of the formation of plastic hinges at the feet of the Platform South East and South West legs. Such plastic hinges have been recorded in the sequential member failure Table 9 for the fully fixed model. Plastic hinges form in chord members 9935 and 9933, at nodes 215 and 213 respectively, which are on the sea-ward side to the environmental approach and hence cause a rotation of the structure into the oncoming seas. In Table 11 the first member failure for the pile restrained model occurs at a design shear load factor, i.e. 1.874, than the fully fixed case, 1.729. The first member failure in the pile restrained case is a chord member in the Platform South West leg between elevation 130 ft and 157 ft above mud line. The differences in the utilization ratios can be seen by comparing Figures 13 and 16 for the fully fixed and pile restrained models respectively. Both figures show that the Platform South East and South West legs are heavily loaded along their whole length, and for the fully fixed case there are very high utilization ratios at the feet of these legs, coloured red, depicting the plastic hinges.

The displaced shape (m) at ultimate load of the fully fixed base and pile restrained structures are shown in Figures 18 and 21 for the NE-SW environmental load approach w.r.t Platform North. The magnification factor for the fully fixed and pile restrained cases are 107 and 175, respectively. From these factors it can be interpreted that the pile restrained model having generally larger displacements than the fully fixed model. The NE-SW load approach is from node 217 to 213, see Figure 4. The final displaced shape of both models is in the general direction of the environmental load approach. The sequential member failure for the fully fixed and pile constrained models are found in Tables 13 and 15, respectively. In this instance the two models start with the same member failures, 7383 and 7787 which are found between elevation 130 ft and 149 ft above the mud line. According to Table 13 the fully fixed model can only sustain another two member failures, 9933 and 9937 which are at the feet of the North East and South West legs, before reaching its ultimate load capacity. This is evidenced in Figure 19, the utilization ratio for the fully fixed model by the red highlighting. The pile restrained model can sustain many more plastic hinges before reaching its ultimate load capacity, which according to Table 15 are all chord and brace members in the 130 ft to 149 ft region above mud line. This failure mechanism is quite different compared with the fully fixed model as evidenced by the utilization ratios in Figure 22. These concentrated set of failed members leads to a bowing in the displaced shape of the structure seen in Figure 21.

A.5 Conclusions

A detailed ultimate strength analysis was carried out on the "Benchmark Platform" as a part of a trial application of the draft guideline: "API RP 2A-WSD, section 17.0". The analysis was performed in three main stages: load generation, linear-elastic analysis, and deterministic collapse analysis.

The main findings of this analysis are summarised in the following.

Load generation	Load was generated using Response Surface Modelling. Three loading directions were considered, namely, 180°, 225°, 270° with respect to the platform North.
Linear Elastic analysis	During this stage of the analysis, structural response to deck loads, gravity, wind loads and environmental loads are evaluated. Mean value displacements, internal forces and global resultant forces were calculated. Again, three loading directions were considered.
Deterministic collapse analysis	A progressive collapse analysis was performed using the VDM technique. Strengths of individual members were calculated using the elasto-plastic beam/column model. The joint capacities were calculated using the UEG mean-value formulae. The three loading directions were again considered, and results for both fully fixed and soil-spring support conditions were obtained. From the 100 year reference level load and the computed ultimate capacity in each direction, the Reserve Strength Ratio (RSR), was calculated. These are shown in Table 16 below.

Wave Approach Direction	Angle of Approach w.r.t Platform North	Reserve Strength Ratio (RSR) (fully fixed at mud line)	Reserve Strength Ratio (RSR) (pile restrained at mud line)
East - West	270°	1.17	1.192
North - South	180°	1.679	1.834
North East - South West	225°	0.935	1.030

Table 16: Reserve Strength Ratio

This shows that under the 100 year reference load, the structure would survive all wave approaches except the Platform North East-South West environmental approach with fully fixed connection between mud line and structure. It has been shown that the redundancy beyond the first plastic hinge is very low under the Platform North-South and Platform East-West loading directions, but much larger under the North East-South West loading. The pile restraints seem to increase the redundancy and reserve strength ratio by a small amount, but have a much more significant effect upon the final displaced shape of the structure, and upon the member failure sequence. When the model is fully fixed at the base, the chord members at the feet of the legs are predicted to fail first, whereas the chord and brace members between elevation 130 ft and 149 ft above mud line are predicted to fail first in the pile spring model.

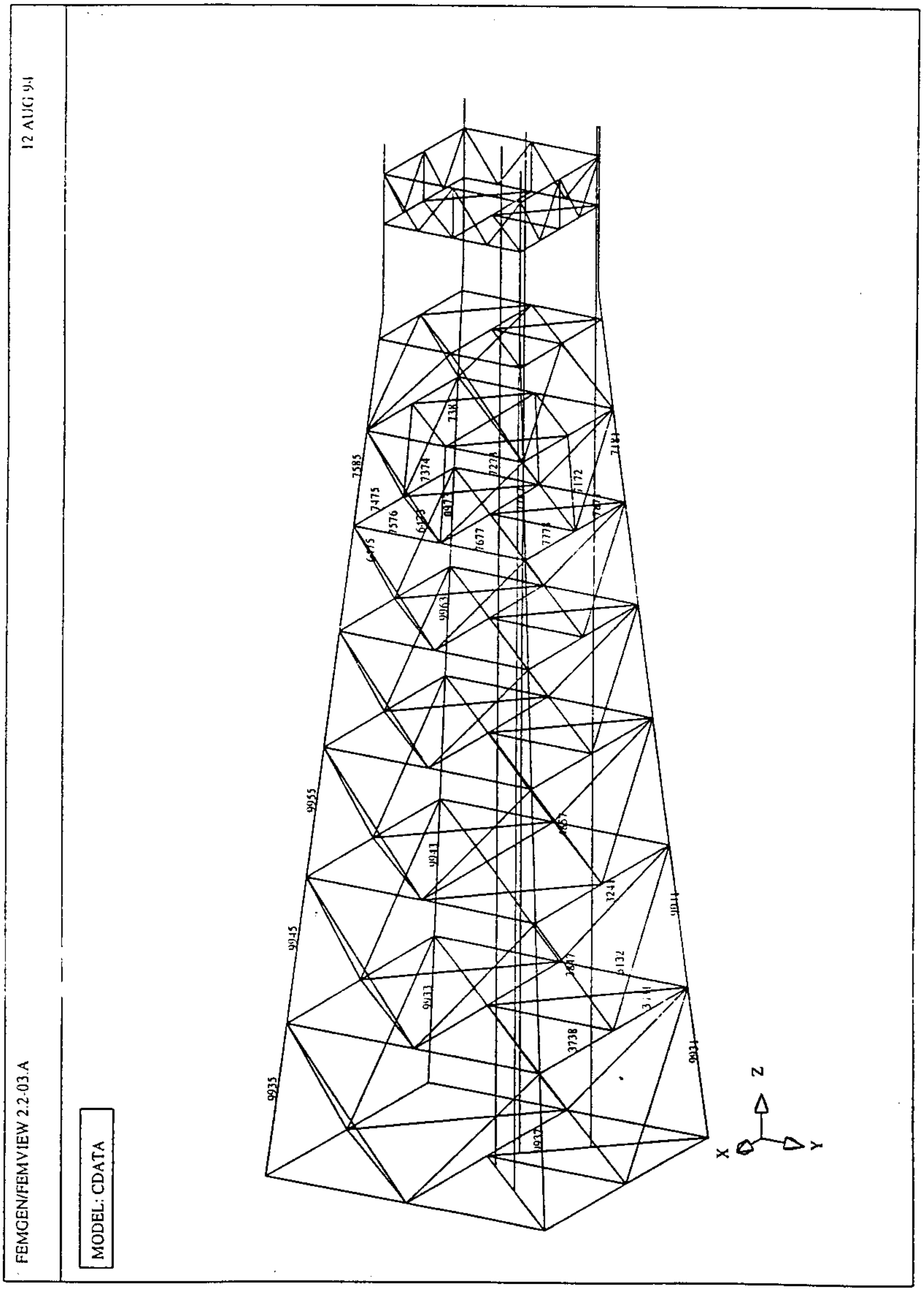
The main differences between the approach used by RASOS and that given by the API-Section 17 code are outlined below.

- There is no facility within the RASOS software system to evaluate the wave/current forces on the platform deck. This, however, is seen to have no influence on the RSR results for the 270° and 180° wave approach directions, but for the 180° direction the RSR is likely to be overestimated.

- The recommended wave model was the 7th order stream function, however RASOS is currently limited to only the linear Airy function.

- The member capacities calculated in RASOS take account of any post-limit behaviour, while the API recommendations do not.

- The joint capacities calculated in RASOS are based on the ultimate strength of the joints, while the API recommendations are based on first cracking.



North East-South West view of Benchmark platform model with member numbers

Figure 3

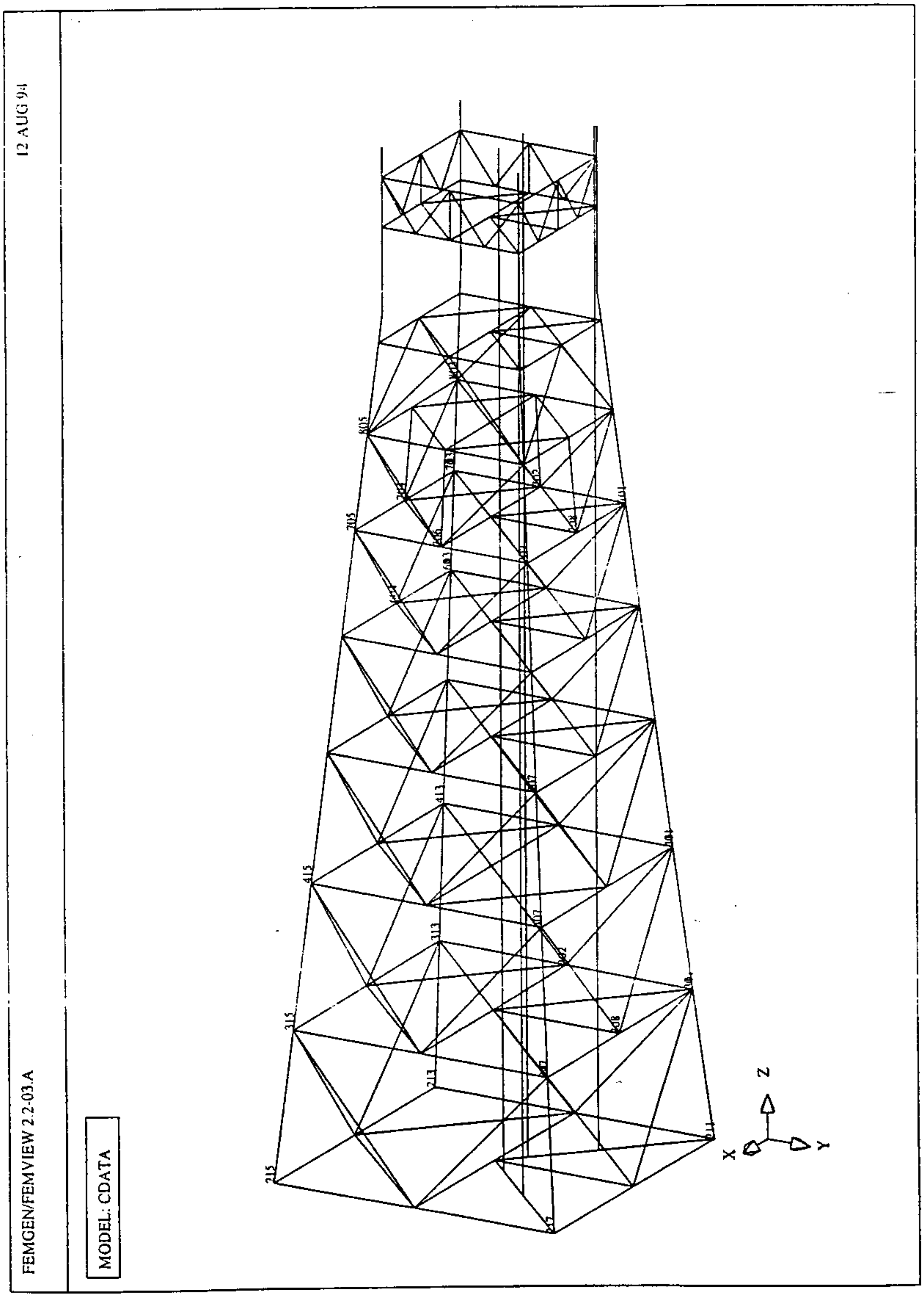


Figure 4

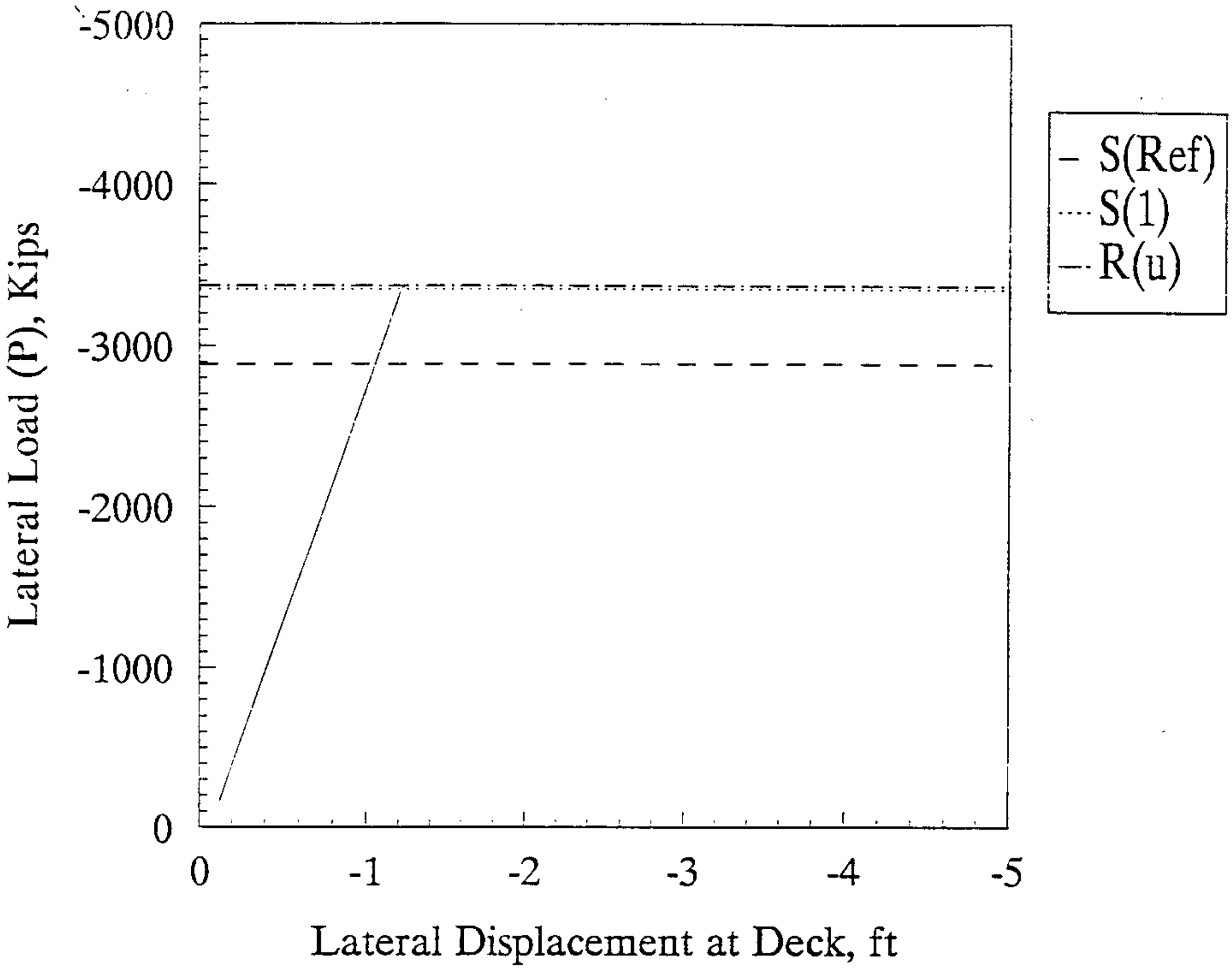
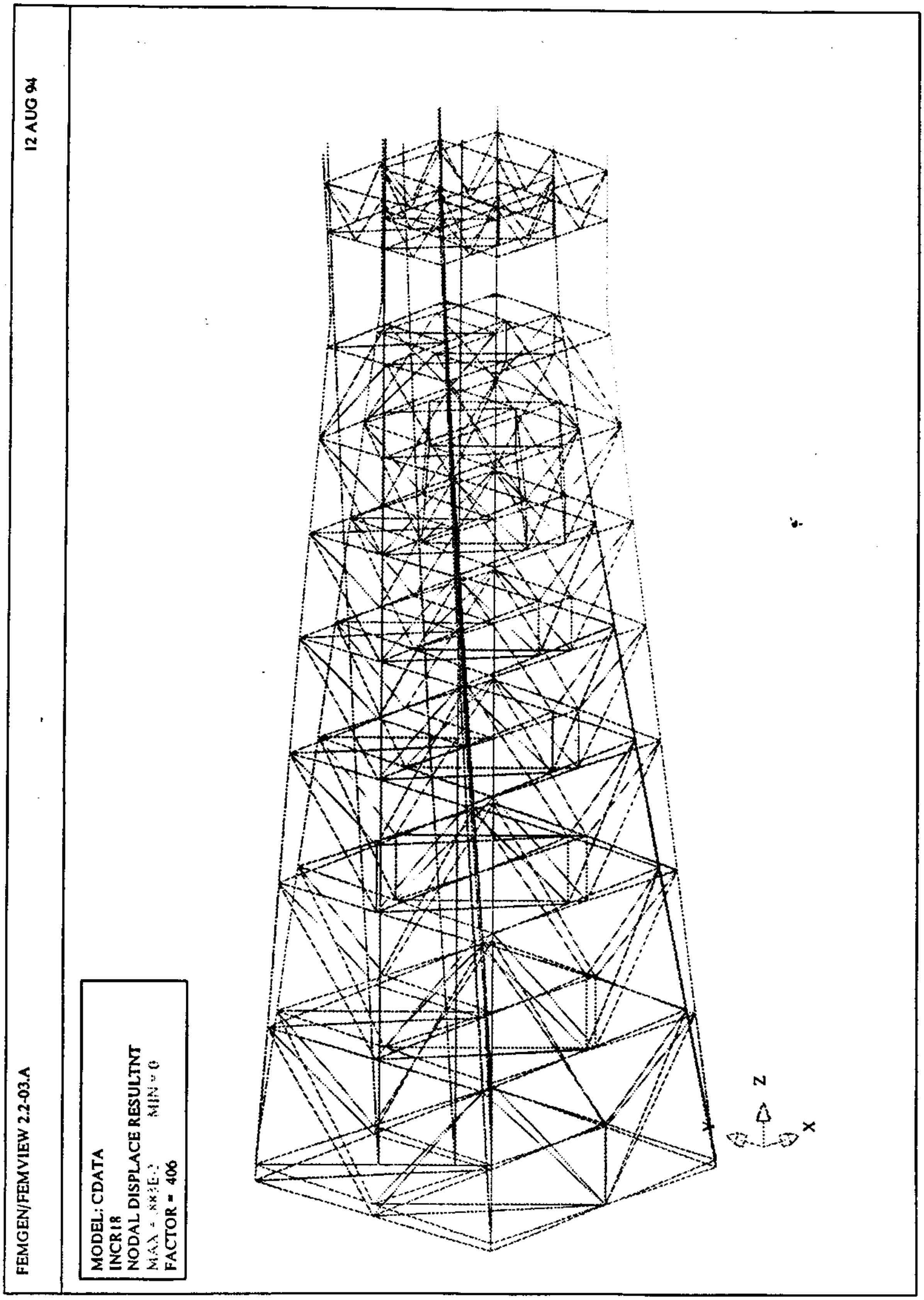


Figure 5: Force-Displacement Curve for East-West Direction (fully fixed at mud line)

Reference Level Load (S_{Ref})	-2883.8 Kips
Load Level at first member failure (S_1)	-3350.9 Kips
Ultimate Capacity (R_u)	-3373.8 Kips
Reserve Strength Ratio (RSR)	1.170
Platform Failure Mode	Combination of legs and braces

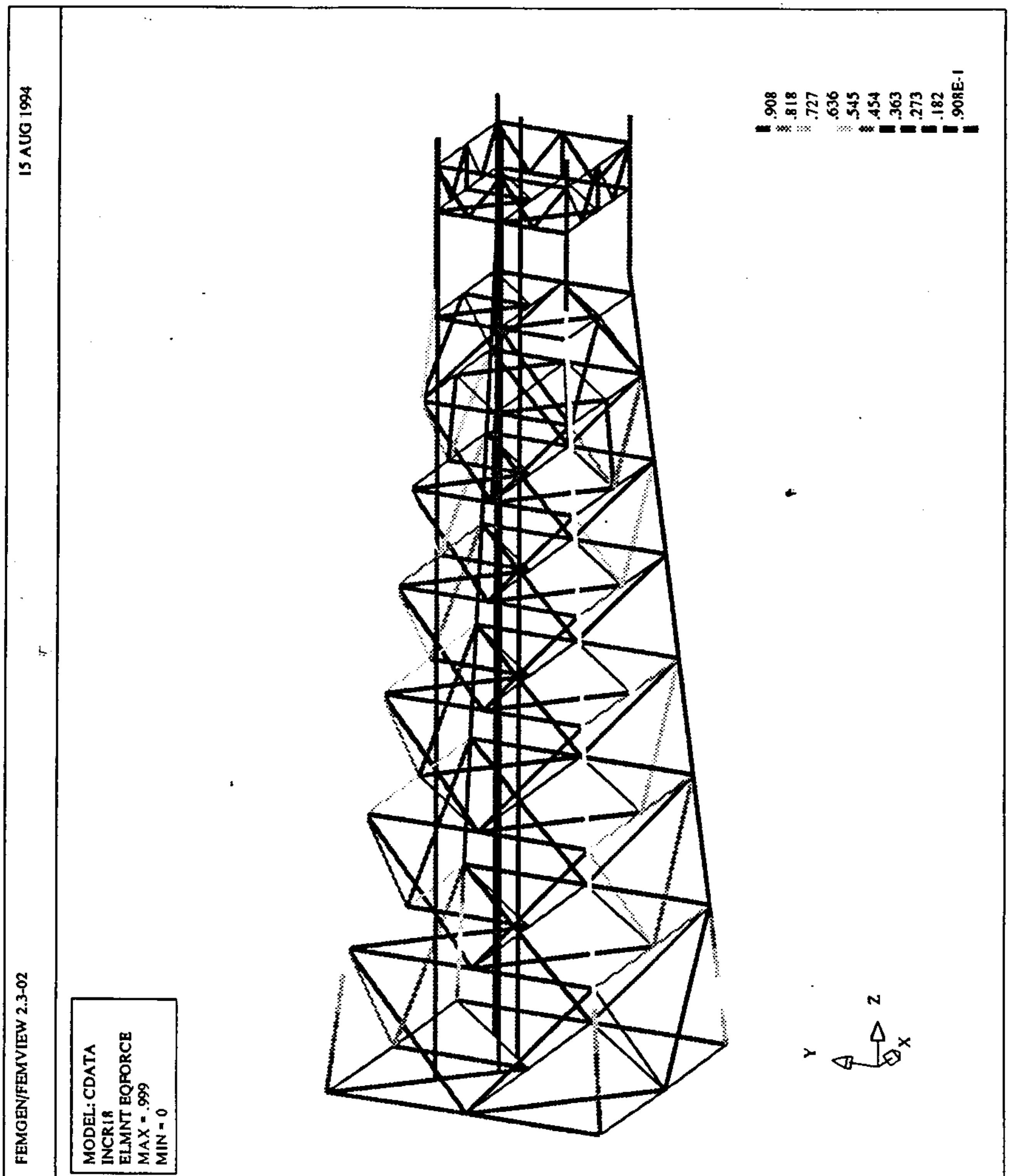


Displaced shape(m) at ultimate load under East-West loading (fully fixed base)

Figure 6

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.123	-161.2	0.056
2	-0.684	-1794.7	0.622
3	-0.871	-2339.2	0.811
4	-1.058	-2883.7	1
5	-1.170	-3210.4	1.113
6	-1.188	-3264.9	1.132
7	-1.198	-3292.1	1.142
8	-1.207	-3319.4	1.151
9	-1.216	-3346.6	1.160
10	-1.239	-3373.8	1.170

Table 4: Ultimate Strength Analysis Results for East-West Direction (fully fixed at mud line)



Equivalent member force at ultimate load under East-West load (fully fixed base)

Figure 7

Member	Node	Load Factor	Base Shear Force (Kips)
9931	211	1.162	-3350.9
9933	213	1.170	-3373.8
9937	217	1.170	-3373.8
9935	215	1.170	-3373.8
9943	313	1.170	-3373.8
9943	413	1.170	-3373.8
3831	301	1.170	-3373.8
7383	803	1.170	-3373.8
9935	315	1.170	-3373.8
9937	317	1.170	-3373.8
7383	703	1.170	-3373.8
3831	308	1.170	-3373.8
9941	311	1.170	-3373.8
3738	308	1.170	-3373.8
3738	307	1.170	-3373.8
Continue	Continue	1.170	-3373.8

Table 5: Member Failure Sequence for East-West Direction (fully fixed at mud line)

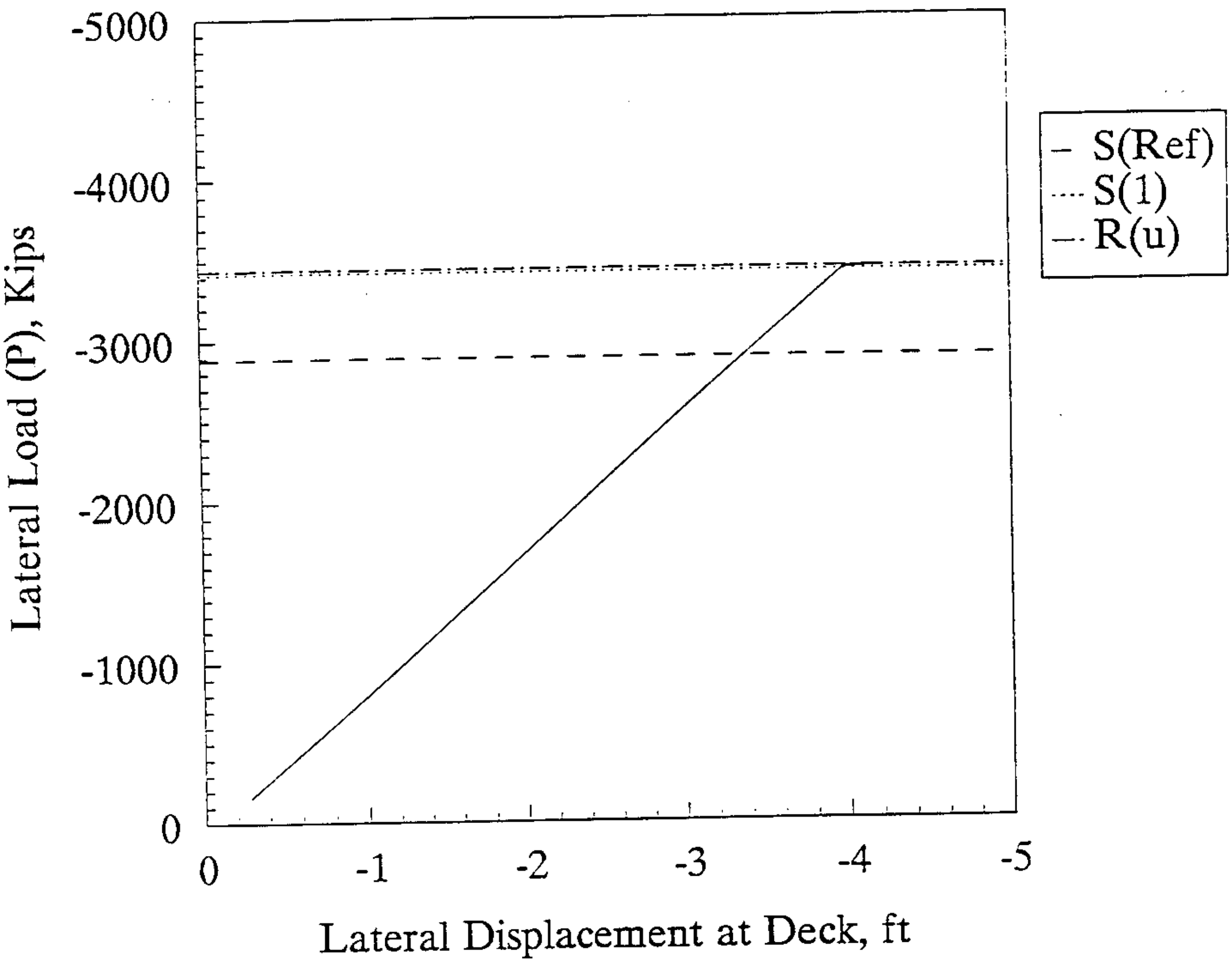
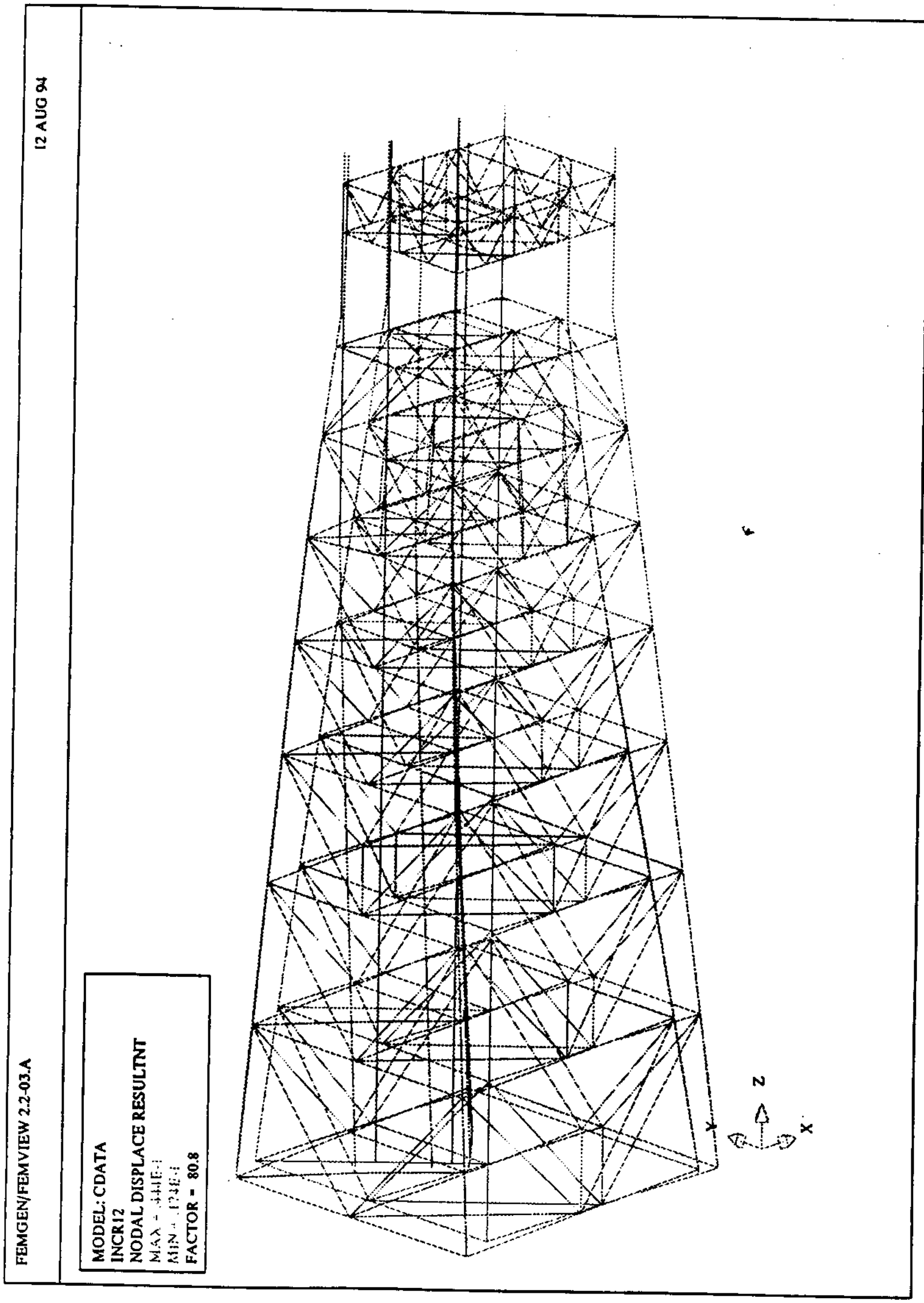


Figure 8: Force-Displacement Curve for East-West Direction (pile springs at mud line)

Reference Level Load (S_{Ref})	-2883.8 Kips
Load Level at first member failure (S_1)	-3417.4 Kips
Ultimate Capacity (R_u)	-3438.9 Kips
Reserve Strength Ratio (RSR)	1.1925
Platform Failure Mode	Combination of legs and braces

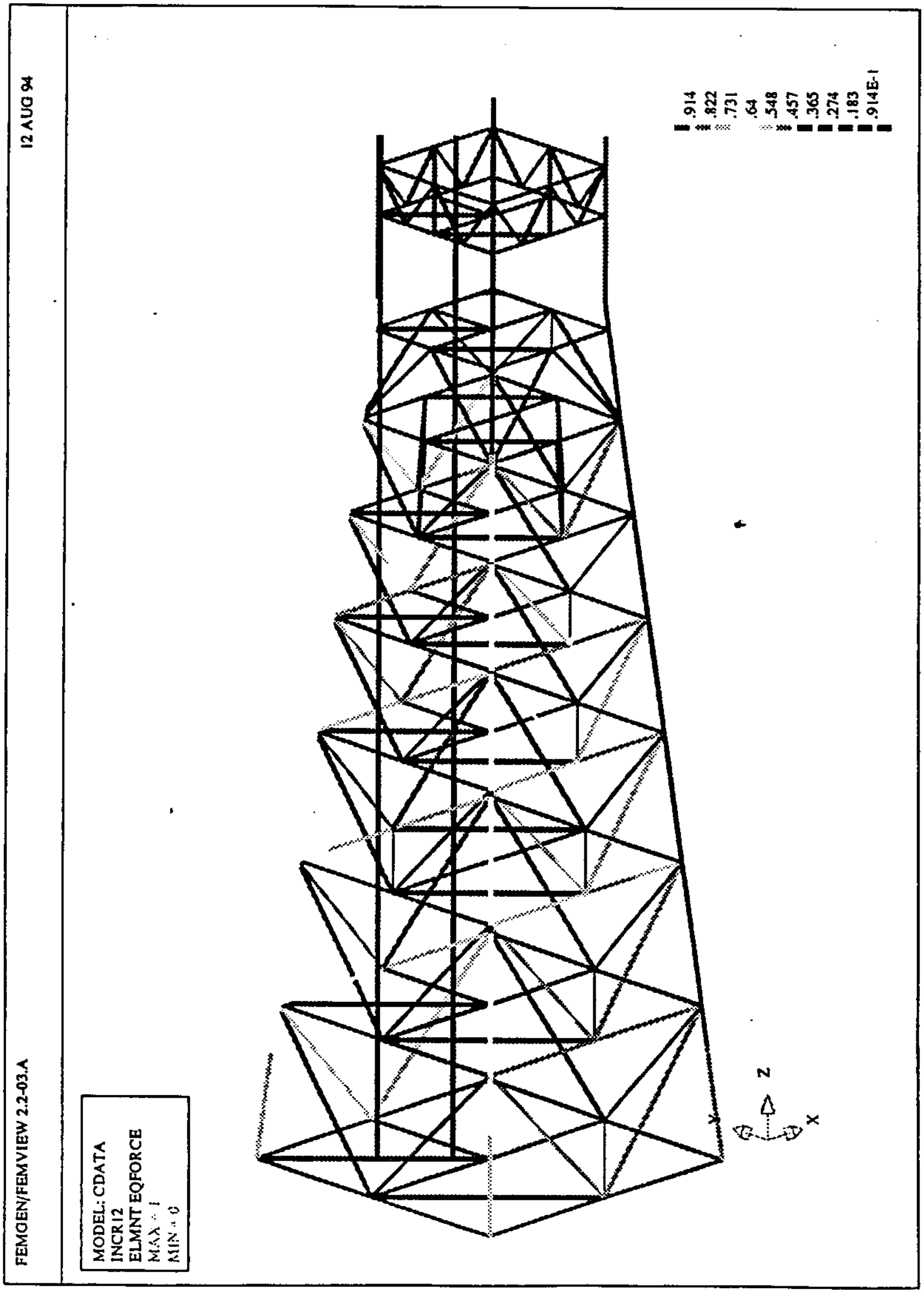


Displaced shape(m) at ultimate load under East-West loading (pile springs)

Figure 9

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.281	-161.2	0.056
2	-1.824	-1522.5	0.528
3	-2.132	-1794.7	0.622
4	-2.441	-2067	0.717
5	-2.749	-2339.2	0.811
6	-3.057	-2611.5	0.906
7	-3.366	-2883.8	1.0
8	-3.674	-3156.0	1.094
9	-3.983	-3428.3	1.189
10	-4.124	-3438.9	1.193

Table 6: Ultimate Strength Analysis Results for East-West Direction (pile springs at mud line)



Equivalent member force at ultimate load under East-West loading (pile springs)

Figure 10

Member	Node	Load Factor	Base Shear Force (Kips)
7181	701	1.185	-3417.4
7383	803	1.190	-3430.7
3847	407	1.192	-3438.9
4857	507	1.192	-3438.9
7585	705	1.192	-3438.9
7475	705	1.192	-3438.9
7475	704	1.192	-3438.9
7576	706	1.192	-3438.9
6475	705	1.192	-3438.9
7576	705	1.192	-3438.9
6373	703	1.192	-3438.9
6473	703	1.192	-3438.9
7383	703	1.192	-3438.9
6373	603	1.192	-3438.9
6473	604	1.192	-3438.9
Continue	Continue	1.192	-3438.9

Table 7: Member Failure Sequence for East-West Direction (pile springs at mud line)

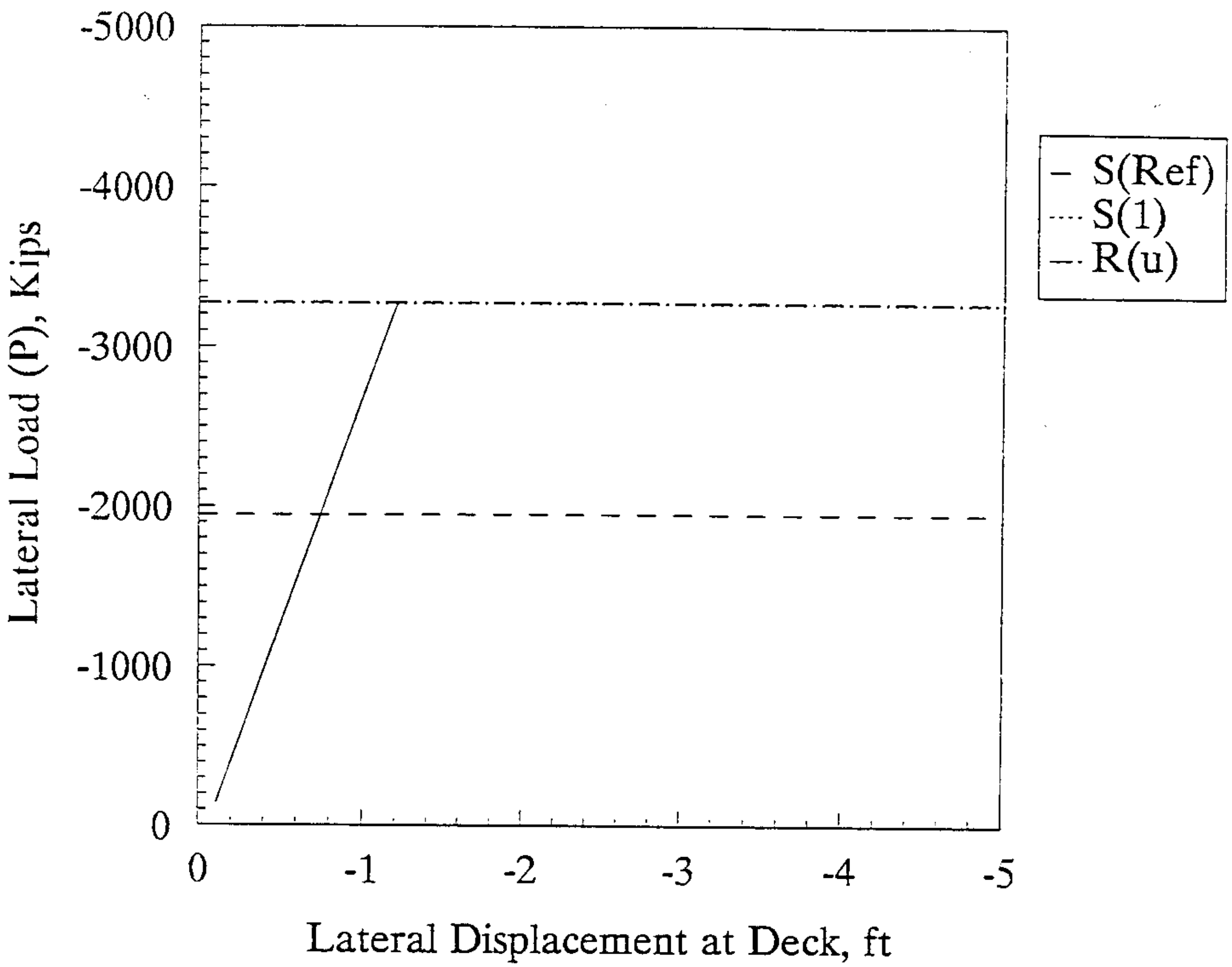
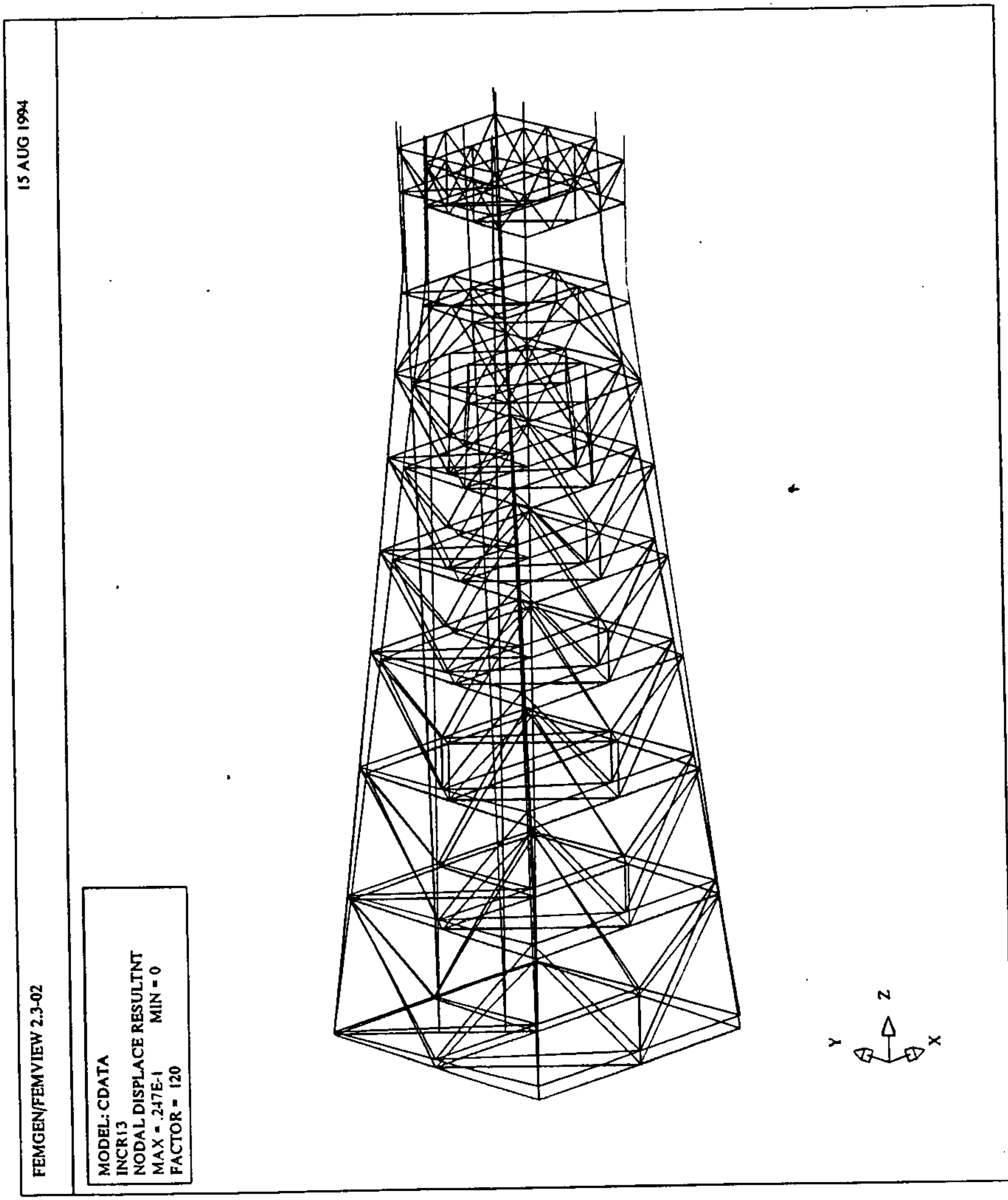


Figure 11: Force-Displacement Curve for North-South Direction (fully fixed at mud line)

Reference Level Load (S_{Ref})	-1947.9 Kips
Load Level at first member failure (S_1)	-3265.2 Kips
Ultimate Capacity (R_u)	-3271.2 Kips
Reserve Strength Ratio (RSR)	1.679
Platform Failure Mode	Combination of legs and braces

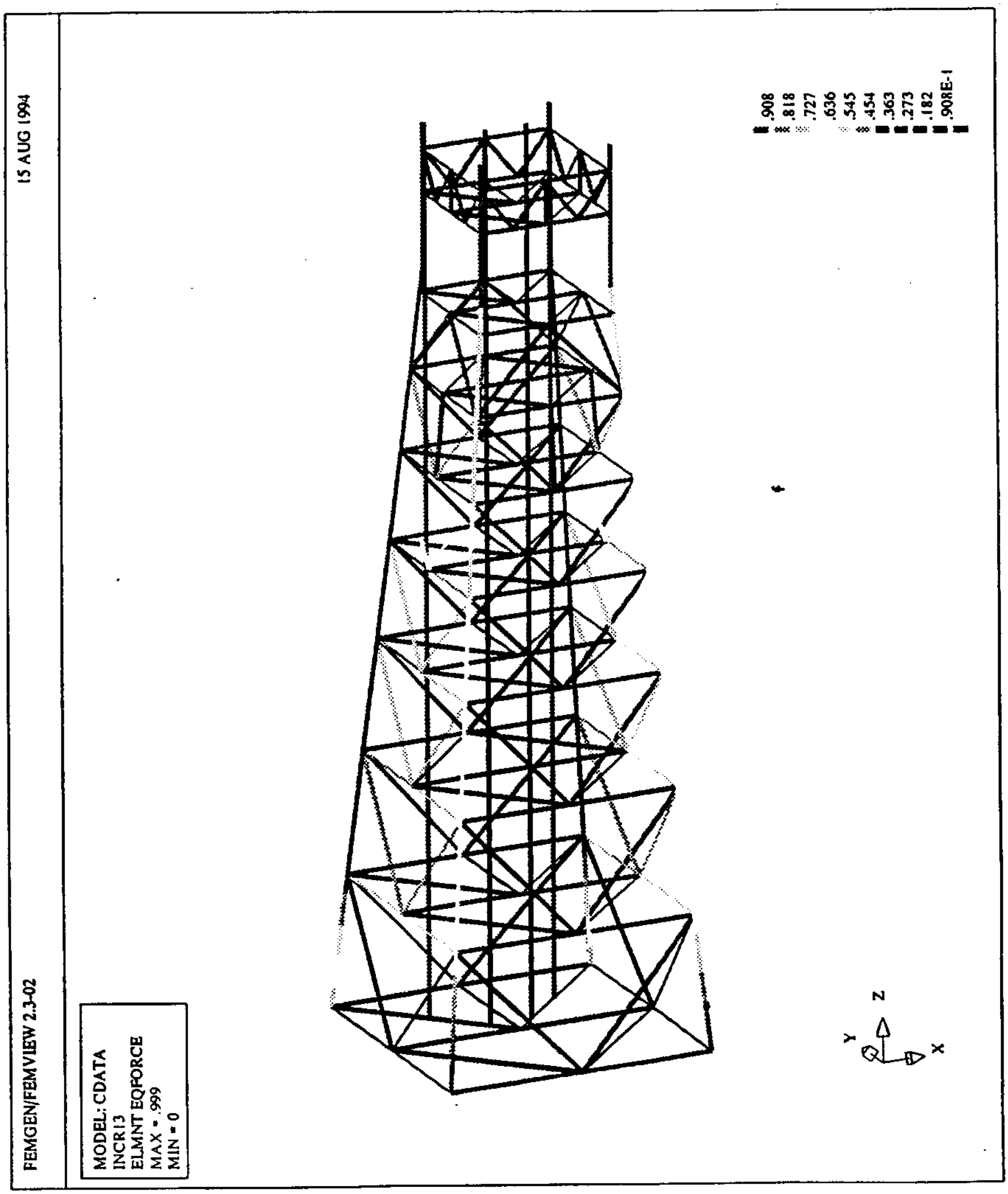


Displaced shape(m) at ultimate load under North-South load (fully fixed base)

Figure 12

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.111	-142.1	0.073
2	-0.426	-1045.0	0.536
3	-0.744	-1947.9	1.0
4	-0.871	-2309.1	1.185
5	-0.934	-2489.6	1.278
6	-0.998	-2670.2	1.371
7	-1.062	-2850.8	1.464
8	-1.125	-3031.4	1.556
9	-1.189	-3211.9	1.649
10	-1.213	-3271.2	1.679

Table 8: Ultimate Strength Analysis Results for North-South Direction (fully fixed at mud line)


FEMGEN/FEMVIEW 2.3-02
15 AUG 1994
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MIN = 0
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.727
.636
.545
.454
.363
.273
.182
.908E-1
Y
Z
X
Equivalent member force at ultimate load, North-South load (fully fixed base)

Figure 13

Member	Node	Load Factor	Base Shear Force (Kips)
9935	215	1.676	-3265.2
9933	213	1.678	-3268.1
9931	211	1.679	-3271.2
9937	217	1.679	-3271.2
9945	315	1.679	-3271.2
3132	301	1.679	-3271.2
3132	302	1.679	-3271.2
9955	415	1.679	-3271.2
9941	311	1.679	-3271.2
9937	317	1.679	-3271.2
9941	411	1.679	-3271.2
9945	415	1.679	-3271.2
9935	315	1.679	-3271.2
9931	311	1.679	-3271.2
9943	313	1.679	-3271.2
Continue	Continue	1.679	-3271.2

Table 9: Member Failure Sequence for North-South Direction (fully fixed at mud line)

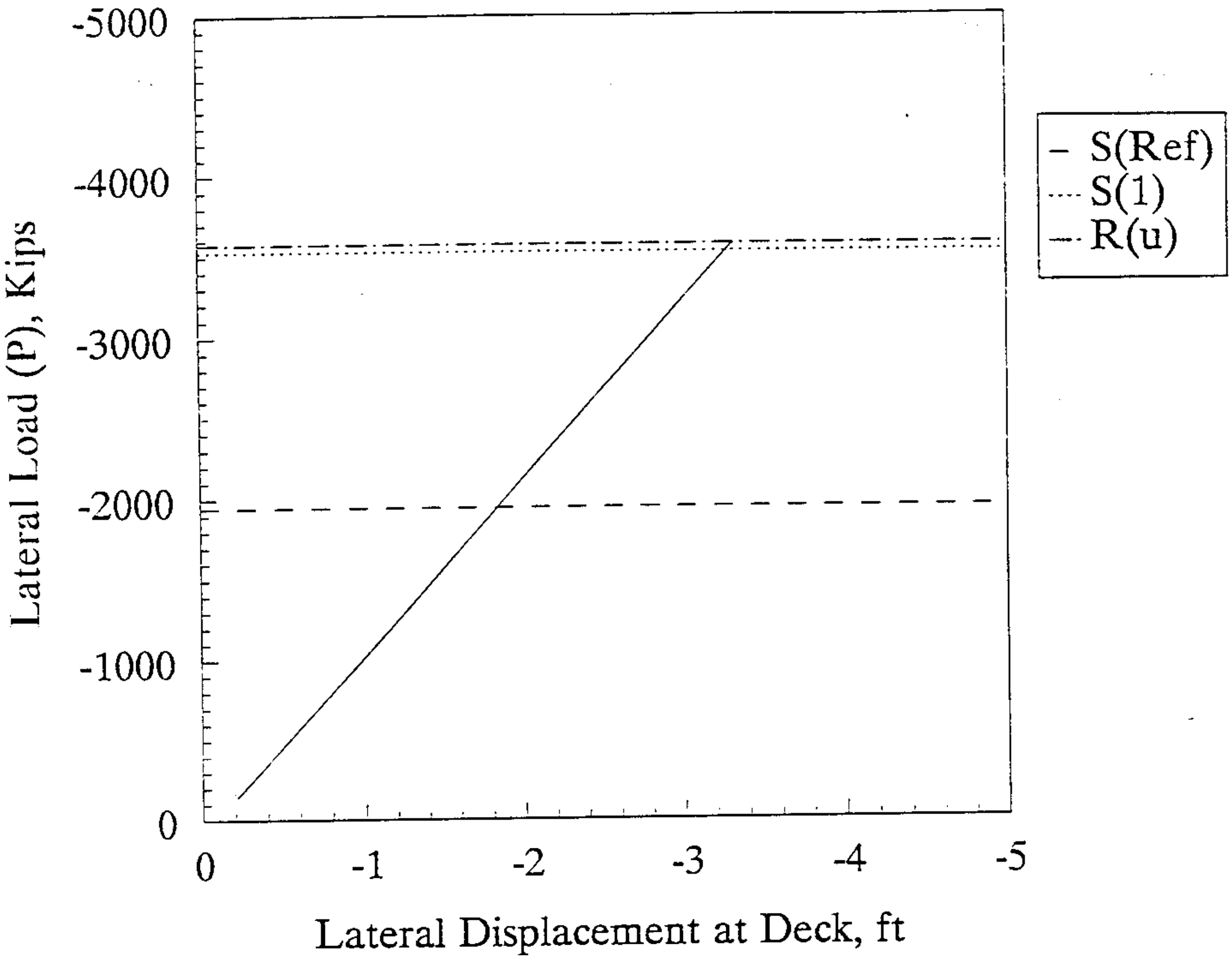
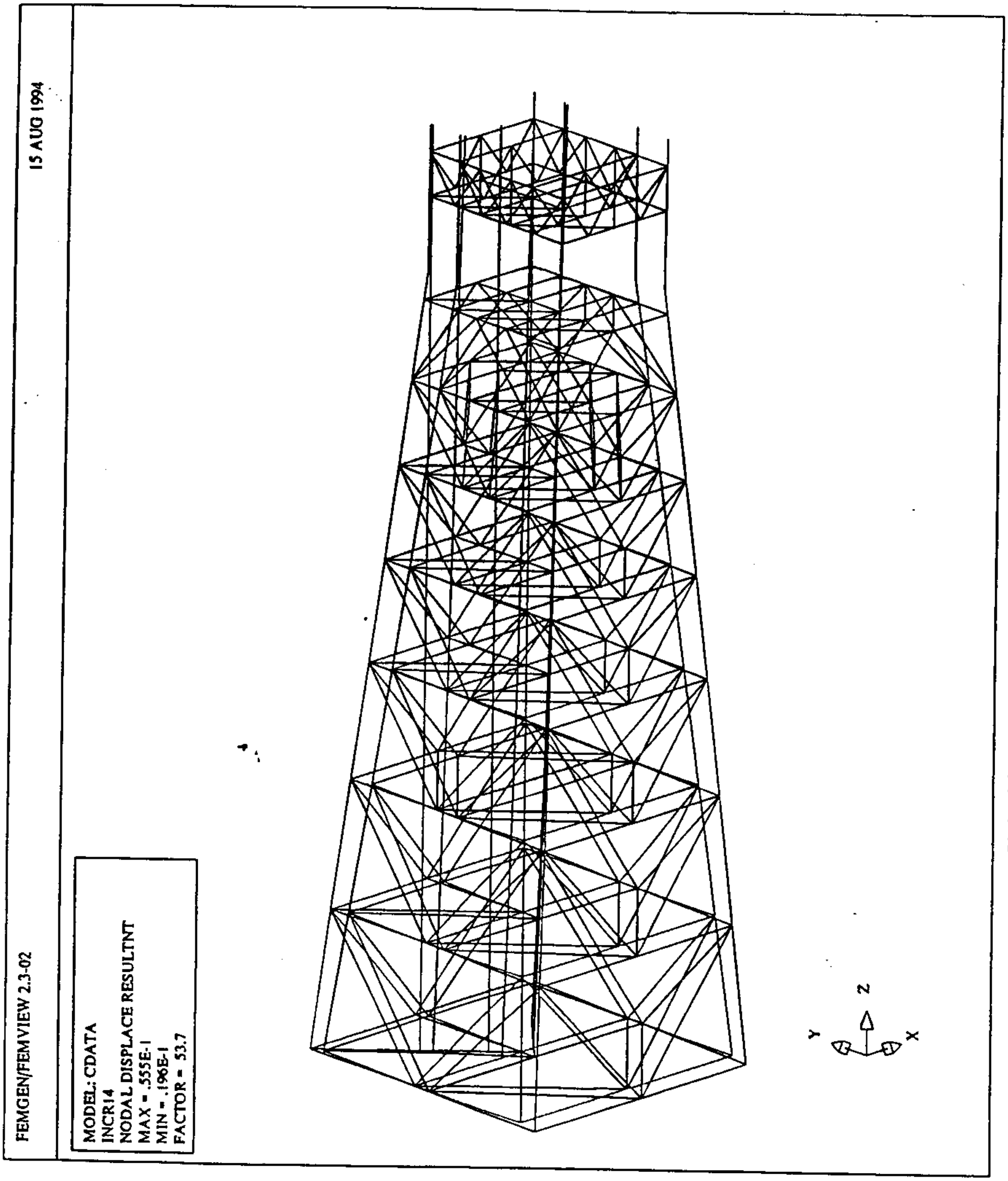


Figure 14: Force-Displacement Curve for North-South Direction (pile springs at mud line)

Reference Level Load (S_{Ref})	-1947.9 Kips
Load Level at first member failure (S_1)	-3527.2 Kips
Ultimate Capacity (R_u)	-3573.1 Kips
Reserve Strength Ratio (RSR)	1.8343
Platform Failure Mode	Combination of legs and braces

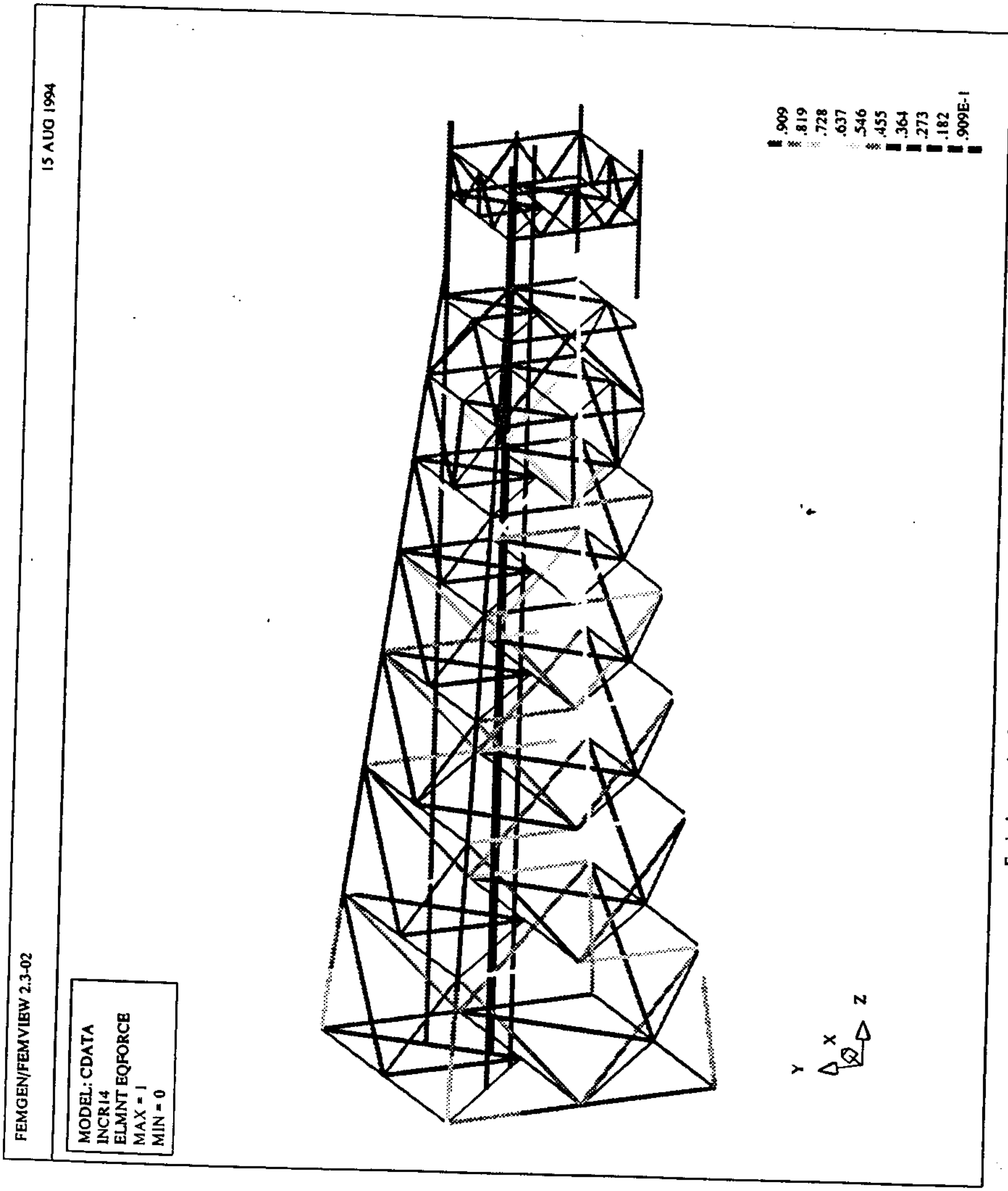


Displaced shape(m) at ultimate load in North-South loading (pile springs)

Figure 15

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.209	-142.2	0.073
2	-1.023	-1045.0	0.536
3	-1.839	-1947.9	1.0
4	-2.166	-2309.1	1.185
5	-2.492	-2670.2	1.371
6	-2.655	-2850.8	1.464
7	-2.819	-3031.4	1.556
8	-2.982	-3211.9	1.649
9	-3.145	-3392.5	1.742
10	-3.310	-3573.1	1.834

Table 10: Ultimate Strength Analysis Results for North-South Direction (pile springs at mud line)



Equivalent member force at ultimate load, North-South load (pile springs)

Figure 16

Member	Node	Load Factor	Base Shear Force (Kips)
7383	803	1.811	-3527.3
7585	805	1.834	-3573.1
3241	401	1.834	-3573.1
7181	701	1.834	-3573.1
7576	705	1.834	-3573.1
7585	705	1.834	-3573.1
7576	706	1.834	-3573.1
7787	707	1.834	-3573.1
6373	703	1.834	-3573.1
7273	703	1.834	-3573.1
7273	702	1.834	-3573.1
7172	702	1.834	-3573.1
7871	708	1.834	-3573.1
7374	703	1.834	-3573.1
7383	703	1.834	-3573.1
Continue	Continue	1.834	-3573.1

Table 11: Member Failure Sequence for North-South Direction (pile springs at mud line)

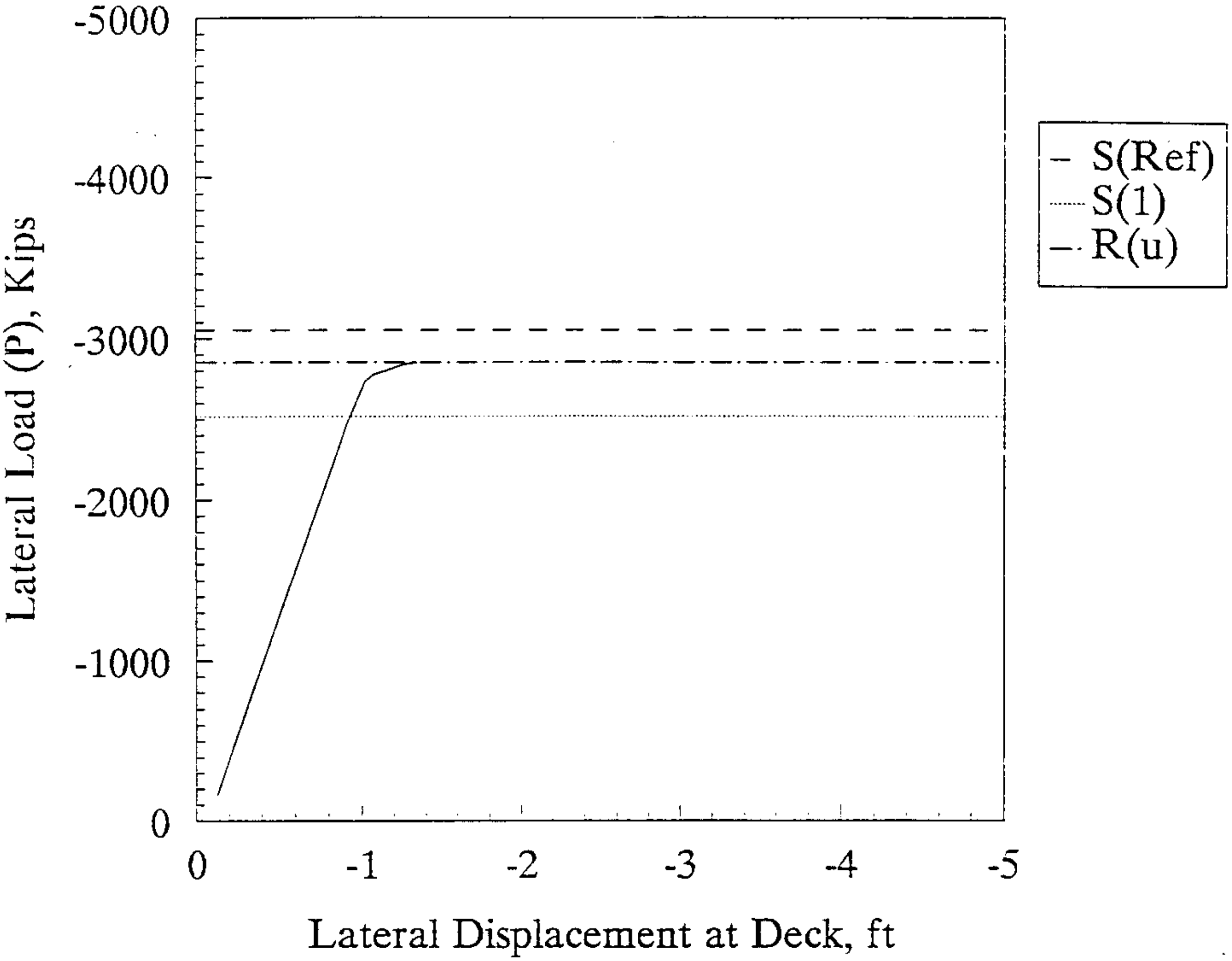
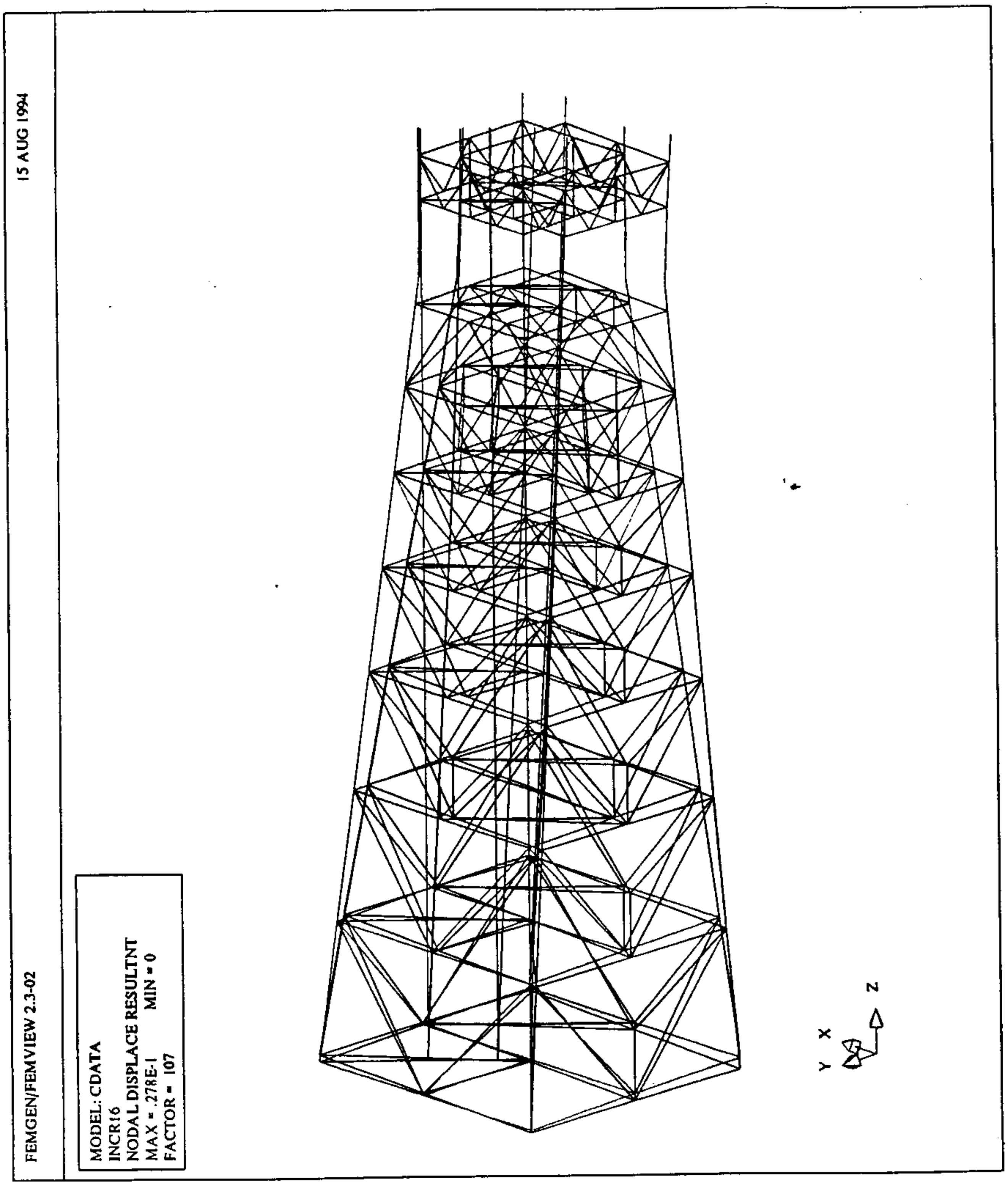


Figure 17: Force-Displacement Curve for North East-South West Direction (fully fixed at mud line)

Reference Level Load (S_{Ref})	-3050.4 Kips
Load Level at first member failure (S_1)	-2591.4 Kips
Ultimate Capacity (R_u)	-2853.2 Kips
Reserve Strength Ratio (RSR)	0.935
Platform Failure Mode	Combination of legs and braces

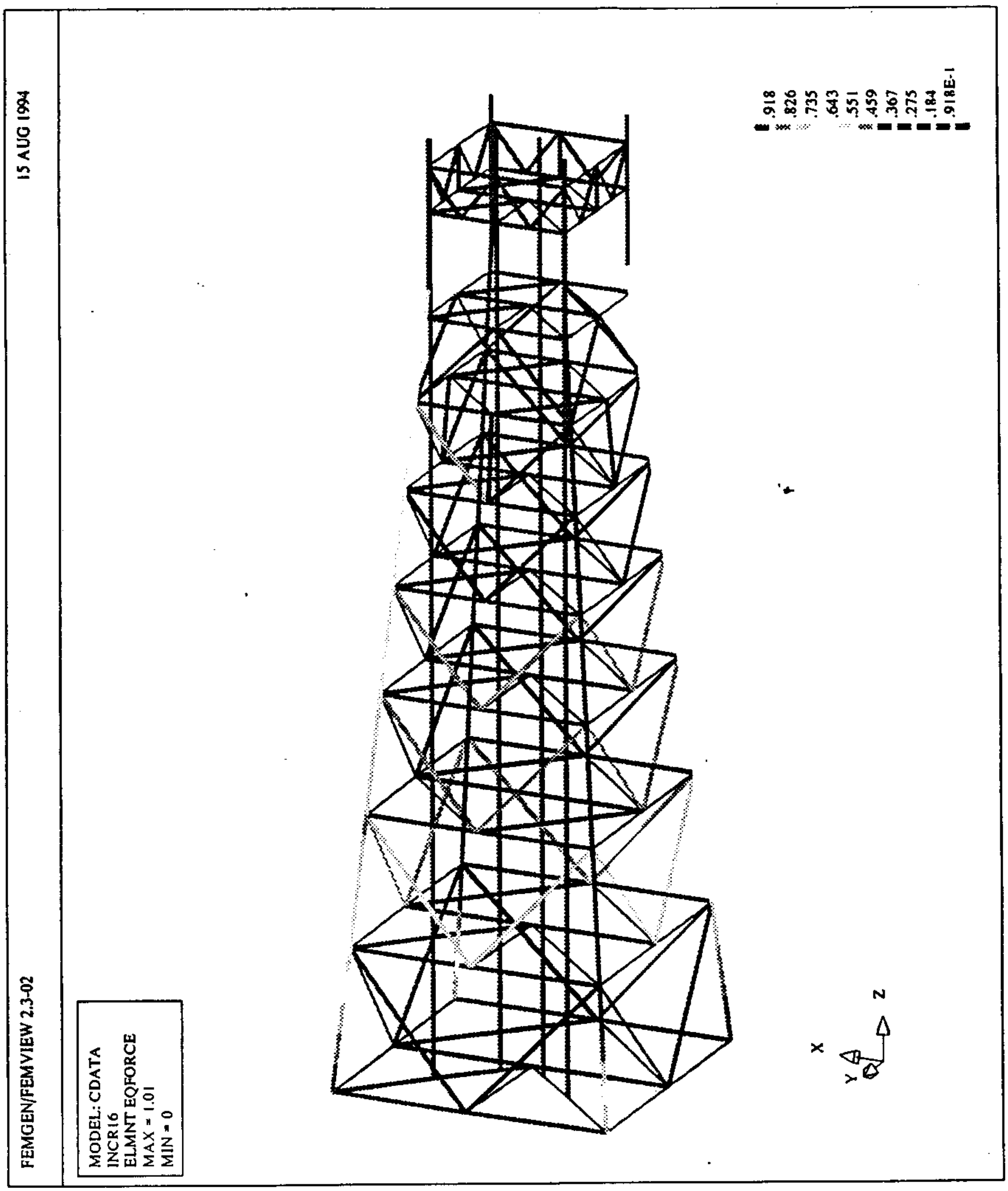


Displaced shape(m) at ultimate load under NE-SW loading (fully fixed base)

Figure 18

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.126	-162.7	0.0533
2	-0.322	-740.2	0.243
3	-0.519	-1317.8	0.432
4	-0.717	-1895.3	0.621
5	-0.914	-2472.9	0.811
6	-1.023	-2734.2	0.896
7	-1.072	-2778.1	0.911
8	-1.163	-2807.6	0.920
9	-1.256	-2837.34	0.930
10	-1.340	-2853.2	0.935

Table 12: Ultimate Strength Analysis Results for North East-South West Direction (fully fixed at mud line)



Equivalent member force at ultimate load under NE-SW load (fully fixed base)

Figure 19

Member	Node	Load Factor	Base Shear Force (Kips)
7383	803	0.849	-2591.35
7787	707	0.885	-2701.1
9933	213	0.915	-2792.1
9937	217	0.931	-2839.1
9931	211	0.876	-2673.6
9935	215	0.876	-2673.6
9943	313	0.876	-2673.6
9943	413	0.876	-2673.6
7273	703	0.876	-2673.6
7374	703	0.876	-2673.6
9937	317	0.876	-2673.6
7383	703	0.876	-2673.6
9931	311	0.876	-2673.6
9935	315	0.876	-2673.6
7677	706	0.876	-2673.6
Continue	Continue	0.876	-2673.6

Table 13: Member Failure Sequence for North East-South West Direction (fully fixed at mud line)

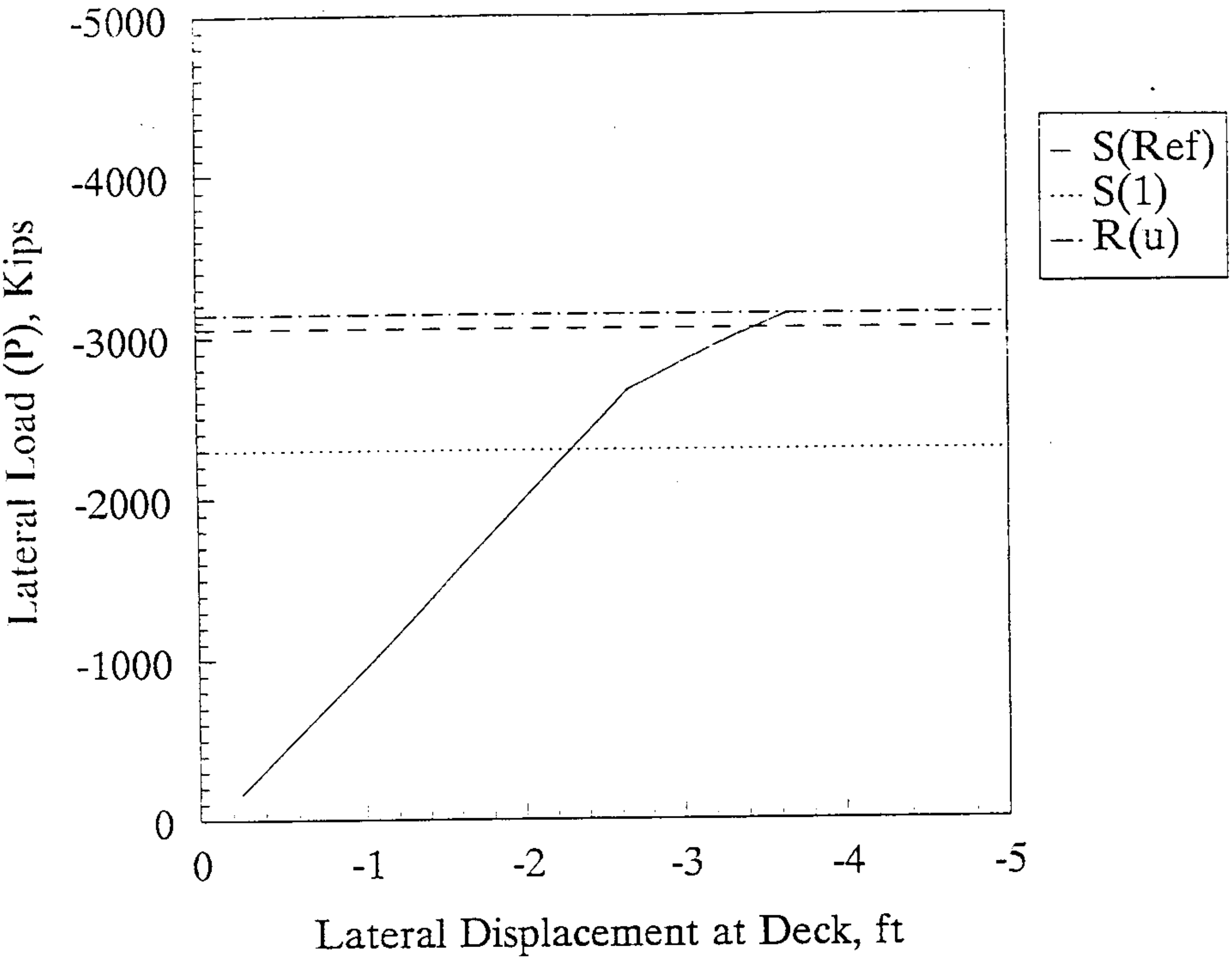
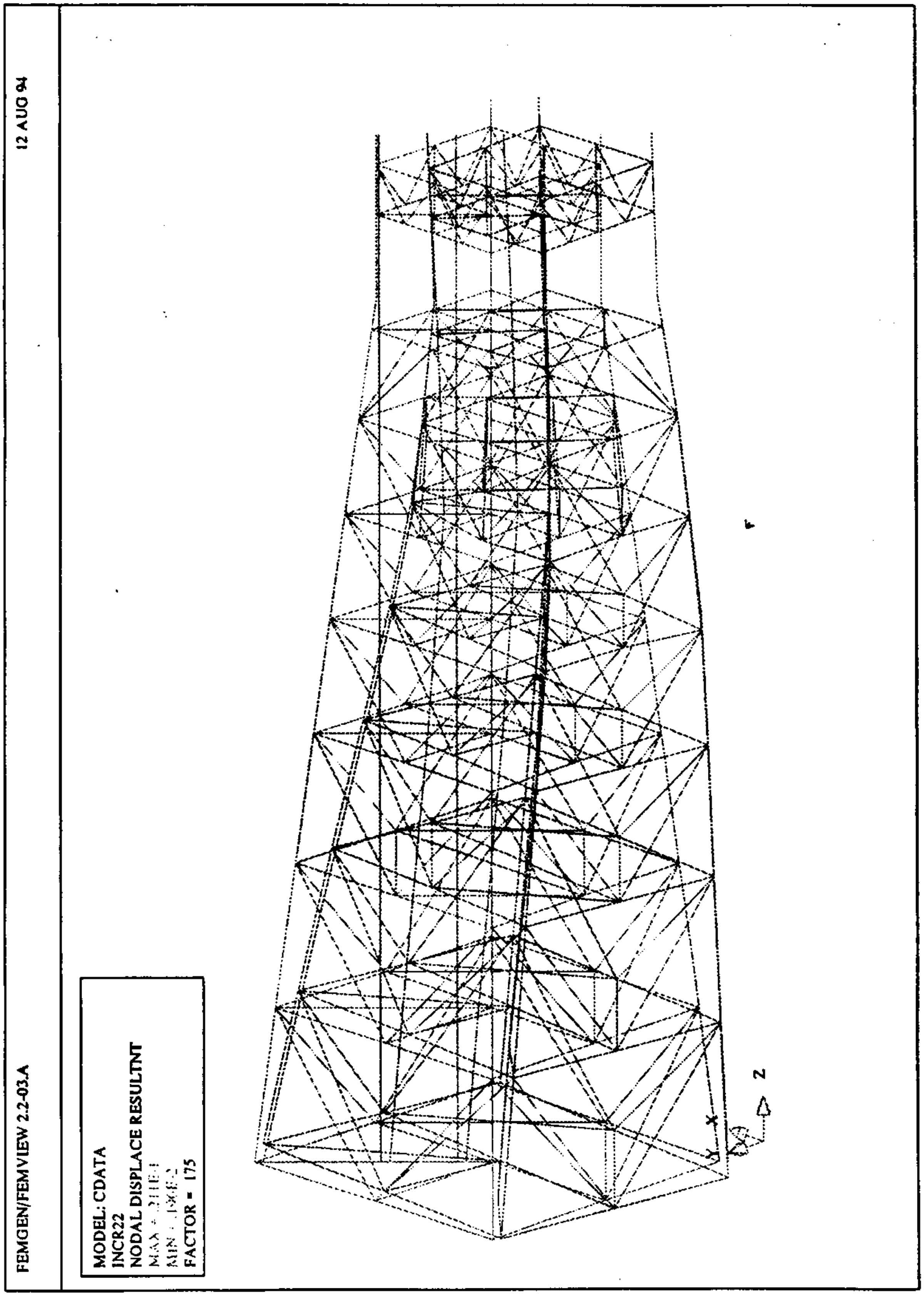


Figure 20: Force-Displacement Curve for North East-South West Direction (pile springs at mud line)

Reference Level Load (S_{Ref})	-3050.4 Kips
Load Level at first member failure (S_1)	-2295.0 Kips
Ultimate Capacity (R_u)	-3142.8 Kips
Reserve Strength Ratio (RSR)	1.030
Platform Failure Mode	Combination of legs and braces

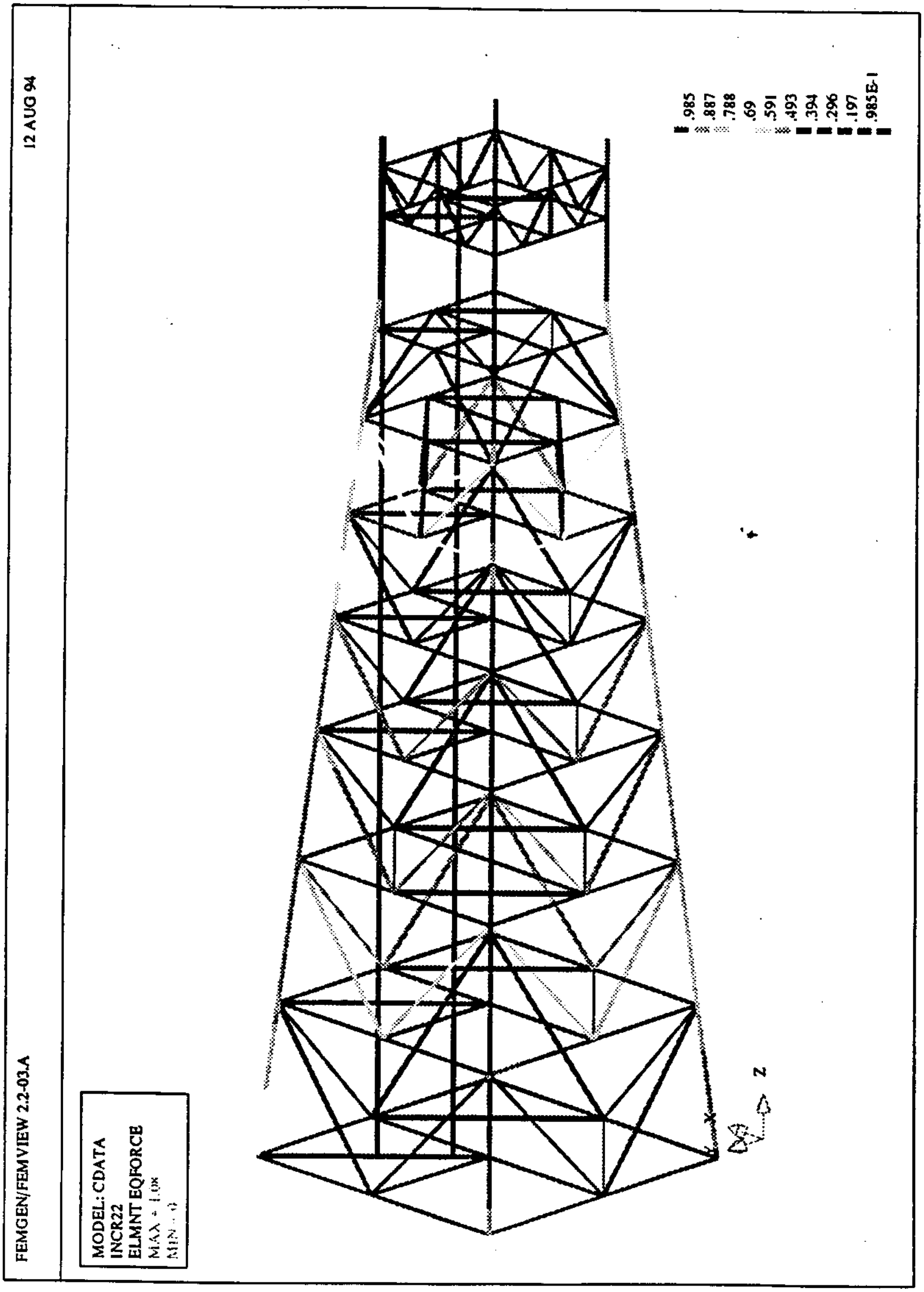


Displaced shape(m) at ultimate load under NE-SW loading (pile springs)

Figure 21

Load Step	Lateral Displacement at Deck Level (+43ft) at South East Leg	Lateral Load (Kips)	Load Factor
1	-0.252	-162.7	0.053
2	-2.649	-2665.2	0.874
3	-2.781	-2733.4	0.896
4	-2.906	-2796.5	0.917
5	-3.030	-2858.7	0.937
6	-3.216	-2952.3	0.968
7	-3.334	-3006.6	0.985
8	-3.438	-3048.1	0.999
9	-3.545	-3094.3	1.014
10	-3.661	-3142.9	1.030

Table 14: Ultimate Strength Analysis Results for North East-South West Direction (pile springs at mud line)



Equivalent member force at ultimate load under NE-SW loading (pile springs)

Figure 22

Member	Node	Load Factor	Base Shear Force (Kips)
7383	803	0.752	-2295.0
7787	707	0.779	-2376.7
6373	703	0.971	-2962.4
7374	704	0.986	-3007.0
7677	706	0.986	-3007.2
7778	708	0.986	-3007.5
7273	702	0.986	-3007.6
7273	703	0.991	-3024.4
7374	703	0.991	-3024.4
7677	707	1.009	-3077.1
7778	707	1.014	-3093.2
9973	713	1.021	-3114.5
9933	213	1.030	-3142.8
7383	703	1.030	-3142.8
9963	613	1.030	-3142.8
Continue	Continue	1.030	-3142.8

Table 15: Member Failure Sequence for North East-South West Direction (pile springs at mud line)

PARTICIPANT "K"

1.0 SUMMARY

This document presents the results of the ultimate strength analyses for the benchmark jacket, undertaken by ' using SAFJAC. The analyses are based on the Benchmark Basis Document from PMB dated 24 February 1994 and the four subsequent revisions.

SAFJAC, for the Strength Analysis of Frames and JACkets, is a specially developed nonlinear analysis program for calculating the reserve and residual strength of framed structures consisting of tubulars, I-beams, box and rectangular sections. The program can predict large displacement behaviour of two and three dimensional frames including the effect of buckling, material plasticity and joint nonlinearity.

SAFJAC was developed as part of the Frames Project JIP with sponsorship from nine operators and the HSE. It has been extensively verified against test data and standard simple solutions and elastic progressive collapse (member replacement) analyses of complex structures. It is now used by to perform progressive collapse analyses for structures under extreme environmental loads or subject to accidental damage or overload.

The SAFJAC computer model was converted directly from a SACS/SEASTATE model developed by to generate structural dead and environmental loads. The model contains all primary jacket members, piles, conductors, risers, boat landing and bumpers, with a simplified deck structure including four horizontal plans and four deck legs. Each member was modelled using a quartic elastic beam-column element to include large deflection behaviour with automatic subdivision either into cubic elastic-plastic elements or plastic hinge elements on detection of plasticity.

Element types used in the benchmark analyses are:

- Quartic elastic beam-column element modelling geometric nonlinearity with automatic subdivision into cubic element on detection of plasticity.
- Cubic elastic-plastic beam column with Gauss integration and monitoring points over the sections at the two Gauss points.
- Quartic elastic beam-column element with automatic subdivision into plastic hinge elements.
- Piecewise linear joint element with decoupled translational and rotational nonlinear springs.

Benchmark ultimate strength analyses were performed for three wave approach directions, at 180°, 225° and 270° with respect to platform North (measured clockwise). A series of analyses was carried out to establish the reserve strength of the platform and five results are presented in detail in this report.

For the 'required' benchmark case, the piles below mudline were modelled using beam-column elements and the axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using spring elements. The jacket was initially modelled with no account of joint flexibility. Results are presented for the critical 270° wave approach direction. The soft foundation allowed large axial and lateral movements of the piles contributing to large deflections at the top of the jacket.

The resistance of the foundations to the conductors was also modelled (with no gap allowance) and this led to local failures in the plan bracing at the mudline (Elevation -157 ft.). Results are presented in Table 1. below.

The voluntary analyses were undertaken for the jacket model rigidly fixed at Elevation -169 ft (ie. below the mudline). This gave the opportunity to explore the dependence of ultimate strength predictions on various modelling assumptions which would otherwise be masked by the dominance of the foundation driven failure for this installation. Four additional analyses are reported here to enable the following comparisons with respect to ultimate capacity and RSR to be made. The results are reported in the table and the figures in brackets indicate the analyses forming the basis of the comparison.

- Variation for different wave approach directions (RJRF - 180°. 225°. 270°)
- Relative influence of the weak foundation (RJRF and RJPF - 270°)
- Relative influence of joint flexibility and limiting capacity (RJRF and FJRF - 270°)
- Influence of nonlinear element modelling (RJRF - 270° - see Section A.5)

Analysis	Required	Voluntary			
	270° wave	180° wave	225° wave	270° wave	
Load level	RJPF	RJRF V3	RJRF V2	RJRF V1	FJRF V4
Ultimate capacity (Kips)	2039	4196	4740	4046	4077
Section 17 Loads (Kips)	2702	1880	2830	2702	2702
Ref. Loads API 20th (Kips)	2268	1590	2420	2268	2268
RSR	0.90	2.64	1.96	1.78	1.80
Note: RJPF - Rigid joints and piled foundation RJRF - Rigid joint and rigid foundation FJRF - Flexible joints and rigid foundation					

Table 1: Summary of ultimate strength and RSR for Benchmark platform

PART A: BENCHMARK ANALYSIS

A.1 Environmental Criteria

Environmental criteria selected are based on the Draft Guidelines (API RP-2A WSD) Section 17. The basis of the environmental loads for each of the three approach directions is provided in Table 2.

Environmental data	Approach directions		
Orientation wrt Platform North	180°	225°	270°
Wave height (ft)	55.83	66.84	66.91
Wave period (sec)	13.5	13.5	13.5
Current profile (ft/sec)	3.76	3.87	3.82
Storm surge (ft)	3.0	3.0	3.0
Wind speed @ 10m above MSL (ft/sec)	143.3	143.3	143.3
Wave load in deck (kips)	39.9	197.7	249.6

Table 2: Environmental data for the benchmark analyses

Wave and current deck forces were determined based on the draft guideline (API RP-2A WSD) Section 17 and the projected areas provided by PMB. Wave crest and current velocities were applied on the projected areas at and below the wave crest based on spreadsheet calculations. A resultant force was calculated for each deck area and applied at the centre. That force was applied to the nodal joint at the incident face. If the deck was completely engulfed the force was distributed equally to joints above and below, otherwise upper joints received proportionately less. Loading on the jacket members was generated automatically using SACS/SEASTATE which was checked against another environmental loading generator, STRUDL-SELOS. The agreement was within 3%.

A.2 3-D Model Generation

The benchmark jacket was modelled using quartic elastic-plastic beam-column elements. Taking advantage of the techniques of automatic mesh refinement employed within SAFJAC and the high order quartic beam element formulation to model beam-column behaviour accurately, each member was modelled using a single quartic element. Extra nodes and elements were introduced on the legs to model leg cans. The model contains all primary and major secondary jacket members, piles conductors, risers, boat landing and bumpers and a simplified deck including four horizontal deck plans and four deck legs.

The piles inside the legs were modelled using quartic elements. Spring elements were introduced between the piles and legs at each of the horizontal plan levels to restrain the lateral movement of the pile to that of the leg. The gaps between legs and piles were ignored.

Well conductors were modelled using quartic elements. Spring elements were introduced between the conductors and conductor guides to restrain the lateral movement of the conductors to that of the jacket. The gaps between conductors and conductor guides were ignored given the uncertainty of gap size and initial conductor position.

For the required ultimate strength analysis, piles and conductors below mudline were modelled using quartic elements with subdivision into plastic hinge elements on detection of plasticity to approximate large displacement plastic behaviour of the piles or conductors as the wave loading is increased in the analysis. The axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using spring elements to account for the soil-pile interactions. The soil P-Y, T-Z and Q-Z data generation was based on the data supplied by PMB and the guidelines in API RP-2A 20th Edition which were implemented by using SACS. In the voluntary analyses piles were assumed to be fixed at Elevation -169 ft and conductors were assumed to be supported vertically at the mudline.

Initial analyses were performed with all joints rigidly connected. The code checking facilities of SAFJAC (to API RP 2A), highlighted early joint failures and to demonstrate the influence of joint flexibility and limiting strength on the system response. K joints were modelled in the last voluntary analysis using nonlinear springs. The joint characteristics were determined using a database of nonlinear joint flexibilities in conjunction with the database and mean capacity equations underlying the HSE Guidance Notes equations published in Offshore Technology Report, OTH 89 308.

The material yield strength (mean) has been taken as 42 ksi for all jacket, deck and foundation members. The Section 17.0 ultimate starnegth loads were applied at nodal points throughout the structure enabling proper account to be taken of incident loads on risers, conductors etc.

A.3 Software Description

SAFJAC was specifically developed for the nonlinear Strength Analysis of Frames and JACkets. The program was developed and enhanced through : Frames Project and is now used by . in assessment of existing structures and in specialist design studies. The program can predict large displacement behaviour of two and three dimensional frames, including the effects of buckling, material plasticity and joint nonlinearity. Its development focused on structural idealisation and efficient nonlinear solutions that do not over-simplify member or joint behaviour and that do not depend on assumptions by the user, for example, of where yielding occurs. Particular features of SAFJAC are itemised below with comments on the use of features in the benchmark analysis following:

- Full member cross-section library including tubulars, I-sections and rectangular hollow sections.
- High order elastic beam-column element requiring only one element over the full member length to model buckling accurately, allowing for the end fixity provided by the structural continuity (ie. no requirement for specification of effective lengths).

- Continuous monitoring for onset of plasticity
- Automatic subdivision of members where plasticity occurs without interruption of solution. Automatic insertion of plastic hinges or distributed plasticity elements on member subdivision.
- Alternative approaches to plasticity depending on sensitivity of solution and need for close monitoring of stiffness changes.
- Ability to control overall structural displacement rather than load in order to follow the post ultimate unloading behaviour.
- Separate definition of joint flexibility and nonlinearity by nonlinear joint springs (up to six degrees of freedom per node).
- Availability of extensive database of nonlinear joint properties.
- Full nonlinear foundation modelling.

Element types

A comprehensive set of elements is available and these are summarised in Table 3.

Element	Description
Type 31	Cubic elastic-plastic beam-column element with Gauss integration and monitoring points over the sections at the two Gauss points.
Type 32	Quartic elastic beam-column element with automatic subdivision upon detection of plasticity. On subdivision, Type 31 elements are inserted in the plastic zones.
Type 33	Quartic elastic beam-column element. For large displacement response of a complete member.
Type 34	Quartic plastic hinge beam-column element with automatic subdivision into two elements if a plastic hinge is detected within the element length.
Type 41	Piece-wise linear joint element with decoupled translational and rotational nonlinear springs.

Table 3: Element types within SAFJAC

Pile-soil interaction

Full pile soil interaction was modelled using beam-column elements for the piles and nonlinear springs to represent the P-Y, T-Z and end bearing soil stiffness. Similar modelling techniques were used for the conductors.

Solution procedure

In the ultimate strength analysis, loading is applied in two stages.

Initial static loading representing the still water condition is applied prior to the application of the proportional load due to environmental conditions, including current, wave and wind loads. This allows the still water condition of the jacket to be initialised prior to the application of any environmental loads. The proportional loads are then applied incrementally until the maximum proportional load factor is reached. The post peak response is obtained by pushing the jacket further until the prescribed displacement on a certain node is reached. To overcome convergence problems in the vicinity of the ultimate load, both load and displacement control of the solution were used.

API joint checks

Tubular joint checks to the 20th Edition of API RP2A can be carried out at any stage of the nonlinear analysis. The joints are automatically classified according to their geometry and the load path. Safety factors are removed and a utilisation of unity indicates that the joint load has reached the API lower bound capacity.

Joint behaviour

Nonlinear joint behaviour and strength can be modelled using piecewise linear springs with differing behaviour in tension and compression. Separate springs may be defined for axial, in-plane and out-of-plane bending behaviour.

A.4 Ultimate Strength Analysis (required)

The full platform model including foundation effects is submitted as the required case. Results are presented for the wave approach direction of 270° with respect to platform North. These are given on the following pages in the form of a load-displacement plot and tabulated data as requested.

Figure 1 gives the overall load deflection response and the required capacity information. Table 2 gives datapoints from which the global response is plotted and the first member failure is indicated. Further failures are more conveniently indicated in Figure 2 which gives the order of hinge formation and the load factor (with respect to the reference level). These sites of plasticity can be linked to the local deformation at the peak load shown in Figure 3.

In the platform model, the piles below mudline were included using beam-column elements and the axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using spring elements. The jacket was initially modelled with no account of joint flexibility. The results are presented for the analysis using element Type 34 which introduces a plastic hinge (with no strain hardening) on detection of plasticity.

A number of modelling variations were also implemented to test the robustness of the solution. An alternative analysis incorporated element Type 31 where automatic subdivision introduces a cubic element (instead of the plastic hinge approximation) to model the gradual spread of plasticity though the thickness and along the member length taking account of strain hardening. A third analysis included the leg extensions below the mudline which had previously been omitted and a fourth adopted cyclic.

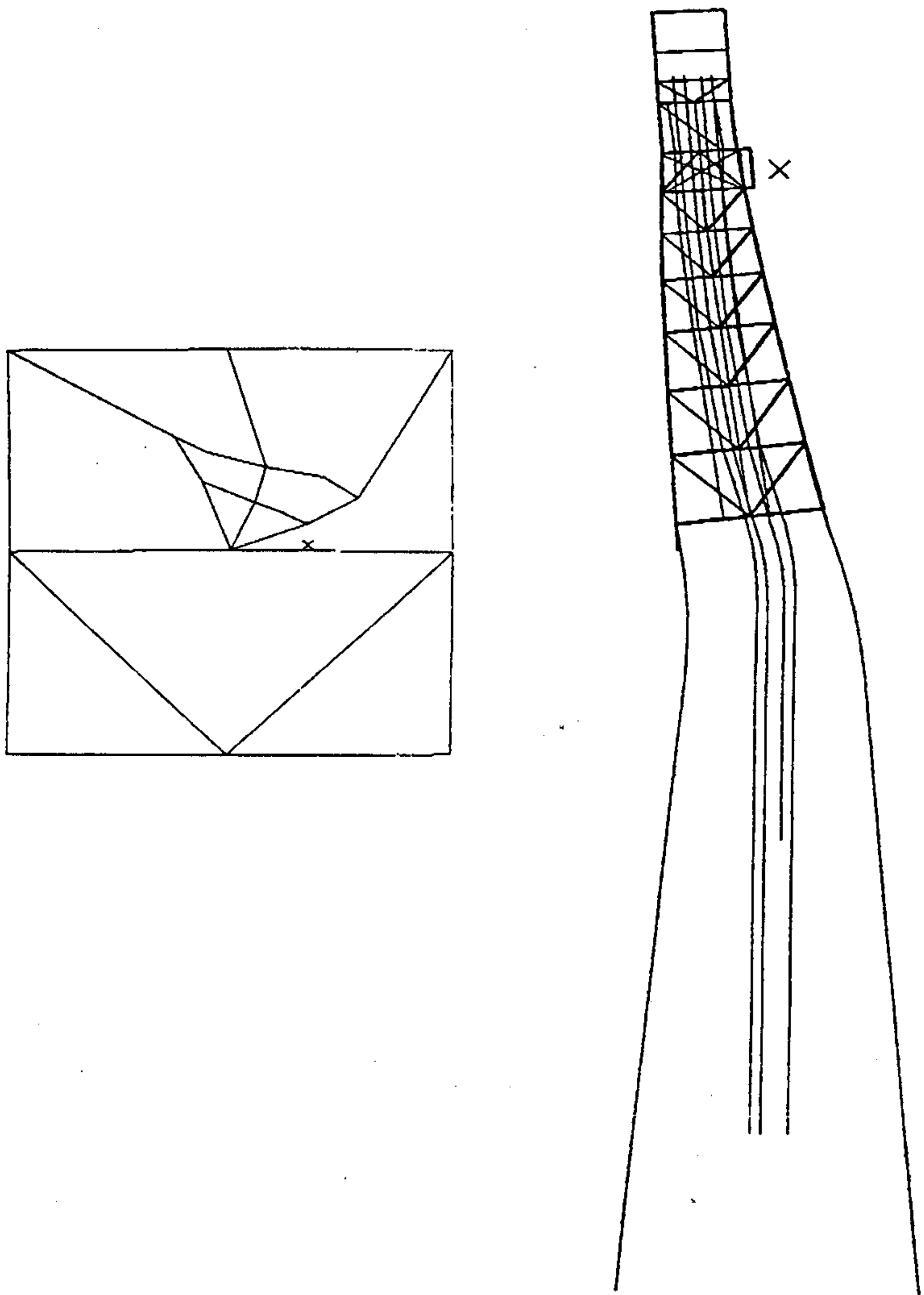


Figure 3: Ultimate Strength Analysis - Minimum Required Results
Deflected shape of jacket and -157 ft plan bracing on attainment of peak capacity for wave approach direction at 270° wrt Platform North (mag. x20)

A.5 Ultimate Strength Analysis (Voluntary)

Additional analyses have been performed for the benchmark jacket. In all cases the foundation characteristics were omitted by fixing the jacket at -169 feet below the mudline. In this way failures associated with the weak foundation were suppressed to enable the ultimate response characteristics of the jacket to be investigated which provides a valuable comparison for benchmark purposes. The other principal differences from the 'Required' case are summarised in Table 5.

Analysis	Difference from 'Required'	Objective
V1	Piles fixed at -169ft. conductors vertically supported at mudline. Quartic elements - subdivision into cubic. Wave approach 270° as 'required' case.	Effect of fixed base/foundation compliance on platform ultimate capacity.
V2	As V1 but wave at 225°	Reserve strength comparisons for alternative wave directions.
V3	As V1 but wave at 180°	
V4	As V1 but K joints modelled using nonlinear springs	Effect of joint behaviour on ultimate capacity.

Table 5: Comparison of voluntary analysis with required case

Analysis V1

Figures 4 to 6 and Table 6 present the results for case V1. Details of component failures are only itemised up to the peak load for clarity.

In this first analysis the joint flexibility is ignored but tubular joint checks to API RP 2A 20th Edition are implemented to check whether the lower bound capacity has been exceeded. Member failures are concentrated in the K bracing in the face frames parallel to the incident wave direction. The response is asymmetric for a number of reasons: the current has a component perpendicular to the wave direction; the conductors are offset to one side of the jacket. As a result initial failures occur in the platform North face. Initial failure occurs at a load level of 3910 Kips (Load factor with respect to reference load of 1.72). Thereafter a series of member failures occur until the ultimate jacket strength is attained at a load factor of 1.78.

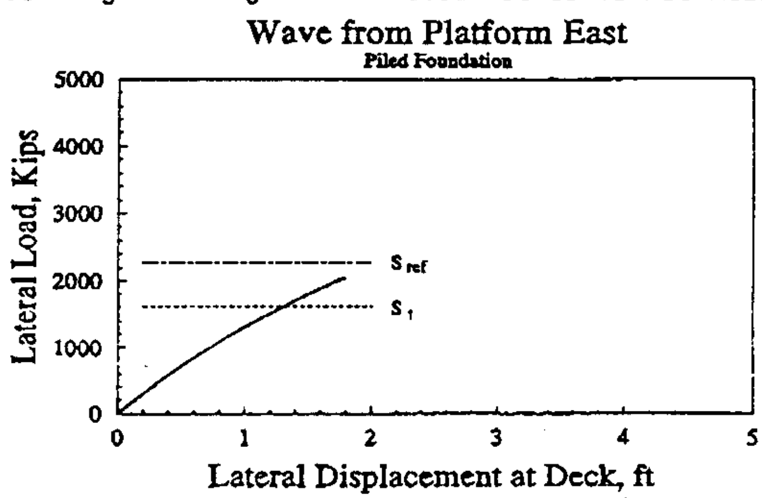
The Type 31 distributed plasticity elements spawned from the initial Type 32 selection successfully model the sequence of failures and the post-buckling curve. An alternative plastic hinge analysis was also performed using Type 34 elements (Table 3). Figure 7 compares the solutions from the two analyses demonstrating the close agreement in the peak load predictions but the post-failure differences arising from the absence of material strain hardening in the idealised plastic hinge approach and the gradual development of plasticity in the cubic elements.

The joint code checks at a load factor of 1.0 (ie. at the reference level load) determine that the loads through the K joints at levels -126ft and -97ft on both face frames and at -71ft on the South frame, exceed the lower bound capacity implicit in API RP 2A provisions. Local increased can wall thicknesses are included in the code checks. The first noncompliance therefore occurs before the initial member failures. The K joints at upper levels become stressed beyond the RP 2A bound with subsequent increases in load. The effect that tubular joint flexibility and limiting capacity may have on the overall response is allowed for in analysis V4.

in place of static. P-Y clay characteristics. In all cases the limit load and general mode of failure remained the same.

The soft foundation allowed large axial and lateral movements of the piles contributing to large deflections at the top of the jacket. The rigid body rotation is a significant component as seen in Figure 3. The resistance of the foundations to the conductors was also modelled (with no gap allowance) and this led to local failures in the plan bracing at the mudline (Elevation -157 ft.). The first hinge forms at a load factor of 0.41 corresponding to a lateral load of 930Kips. Load factors shown relate to the reference level load, although the Section 17.0 loads were applied and factored in the analaysis. A further four hinges occurred by the load factor of 0.70 and at 0.71 the first member has double hinges (hinges 3 and 6) and therefore fails in compression as shown in Figure 2. The maximum load factor (RSR) predicted is only 0.899, at which point a total of 16 hinges have developed in the plan at Elevation -157ft.

Figures 2 and 3 demonstrate the extent of failure in the plan and tubular joint code checks indicate that the connections would also fail. The load factors corresponding to the API RP 2A lower bound are shown in Figure 2. The sites of overstress in members and joints are similar, indicating the inadequacy of the bracing system to sustain the loads/deformations associated with the conductor attachments. Additional non-compliances can be found in the plan connections in the bay above at the end of the analysis. These code checks have been based on the member properties because of the lack of clarity in the drawings. Any can details which would strengthen the joints have therefore been ignored. Even without explicit account of the tubular joints, the platform fails to sustain the assessment load required by Section 17.0 to demonstrate fitness-for-purpose. This is driven by the soft foundation and for that reason detailed modelling of the joint characteristics is not meaningful.



Ls17u - Sec. 17.0 Ultimate Strength Load	2702 Kips
Reference Level Load	2268 Kips
Load Level at First Member Failure	1605 Kips
Ultimate Capacity (Ru)	2039 Kips
Reserve Strength Ratio	0.899
Platform Failure Mode	Jacket

Figure 1: Ultimate Strength Analysis - Minimum Required Results
Load displacement and failure mode results
for wave approach direction at 270° wrt Platform North

Lateral load level at failure of the first member by yielding in the mudline plan bracing 1605 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.382	553		
3	0.703	968		
4	0.943	1245		
5	1.280	1605	1	Yield (Hinges 3 and 6)
6	1.546	1799		
7	1.718	1938		
8	1.788	2039		

* See Figure 2 indicating location and sequence of component failures

Table 4: Ultimate Strength Analysis - Minimum Required Results
Load displacement and failure mode results
for wave approach direction at 270° wrt Platform North

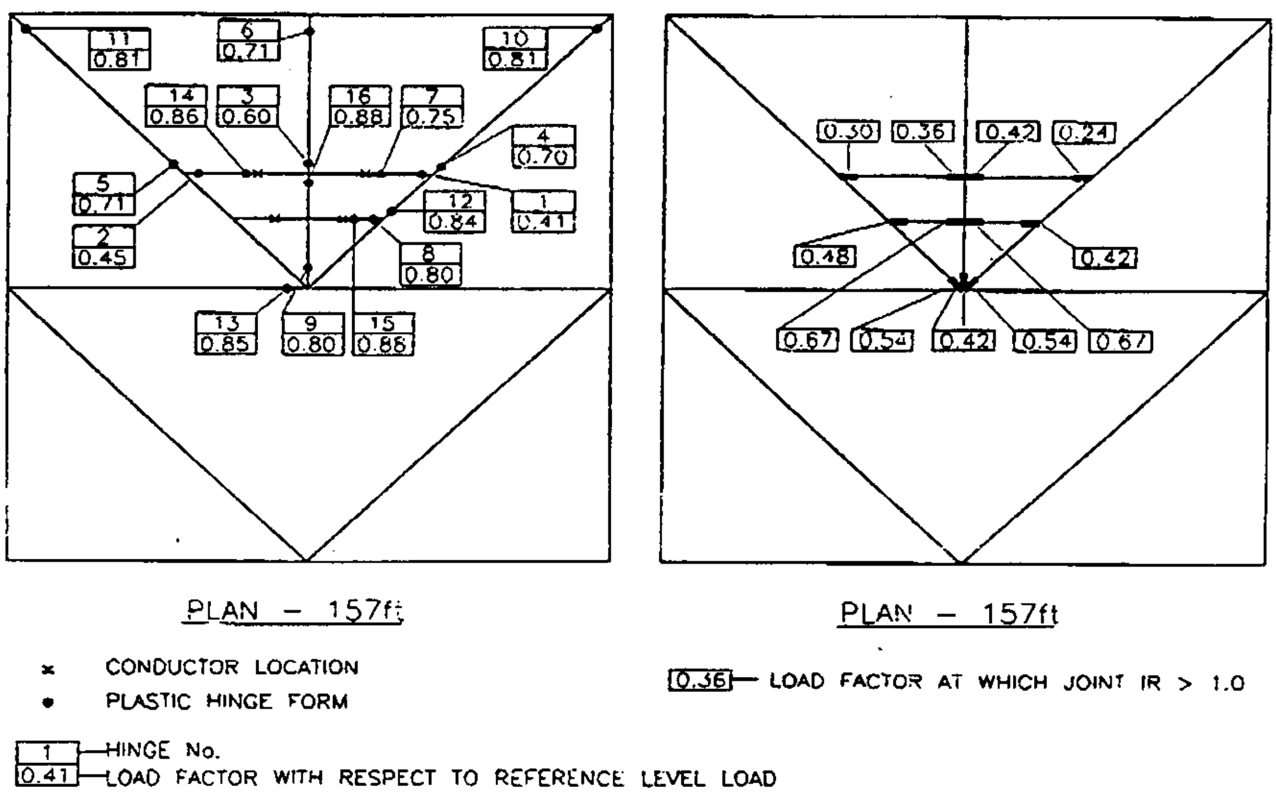
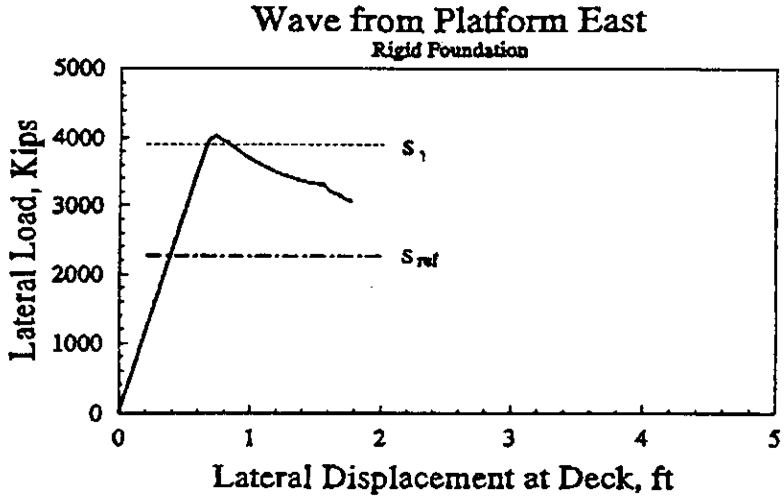


Figure 2: Ultimate Strength Analysis - Minimum Required Results
Location and sequence of component failures for wave approach
direction at 270° wrt Platform North



Ls17u - Sec. 17.0 Ultimate Strength Load	2702 Kips
Reference Level Load	2268 Kips
Load Level at First Member Failure	3910 Kips
Ultimate Capacity (Ru)	4046 Kips
Reserve Strength Ratio	1.784
Platform Failure Mode	Jacket

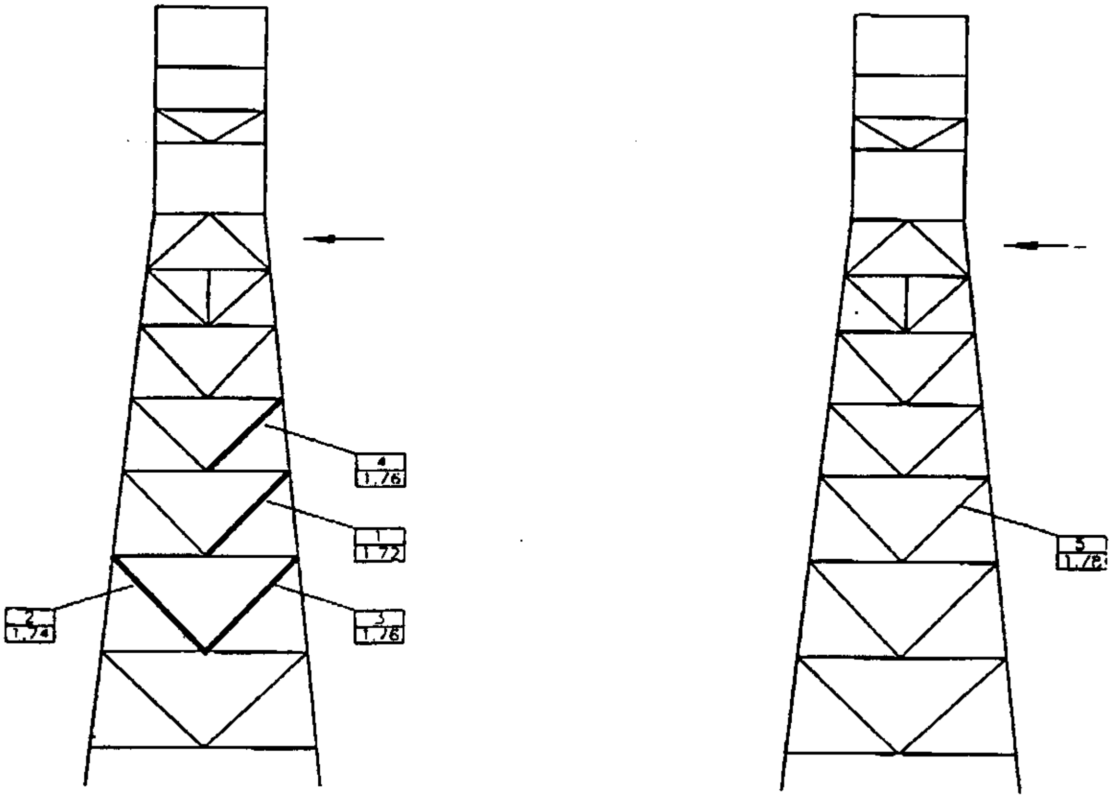
Figure 4: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V1) for wave approach direction at 270° wrt N

Lateral load level at failure of the first member is 3910 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.469	2768		
3	0.610	3598		
4	0.664	3910	1	Buckle
5	0.672	3944	2	Yield in tension/bending
6	0.683	3989	3 / 4	Buckle
7	0.731	4046	5	Buckle
8	0.916	3799		
9	1.102	3581		
10	1.227	3470		
11	1.771	3058		

* See Figure 5 indicating location and sequence of component failures

Table 6: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V1) for wave approach direction at 270° wrt N



Platform North frame

Platform South frame

Figure 5: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case with no joint flexibility (V1) for wave approach direction at 270° wrt N

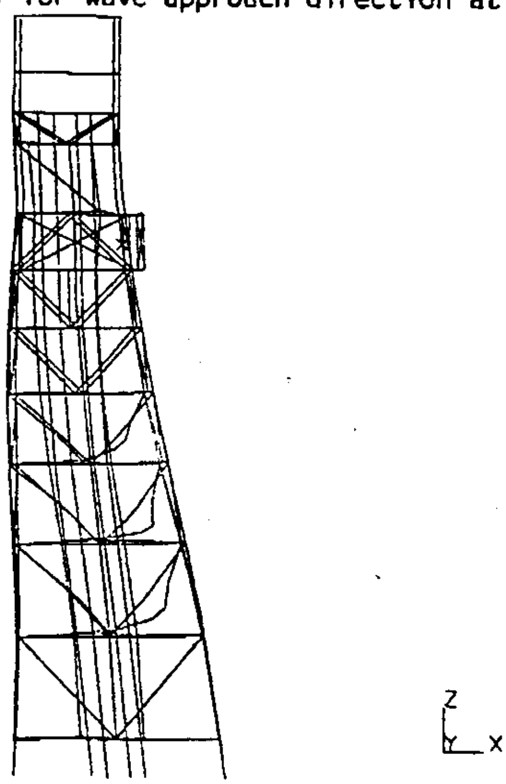


Figure 6: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case with no joint flexibility (V1) for wave direction at 270° wrt N (mag. x 20)

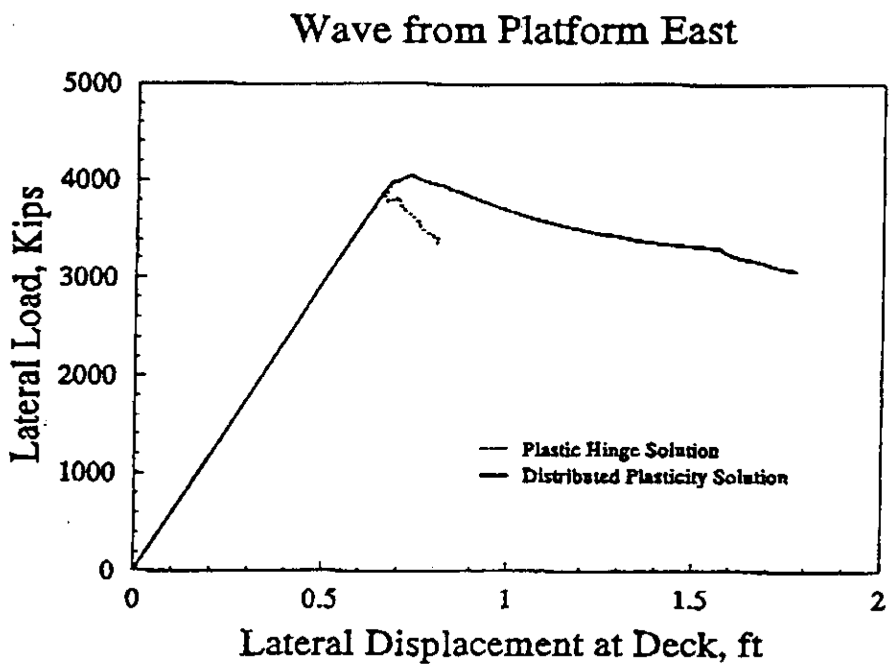


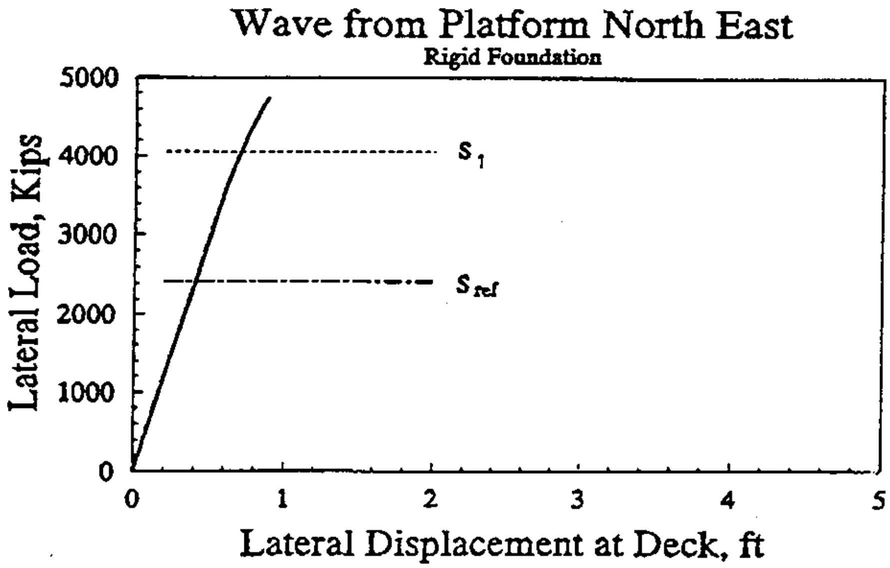
Figure 7: Ultimate Strength Analysis - Voluntary Results.
Comparison of load deflection responses for plastic hinge and distributed plasticity modelling of jacket members for rigid foundation case with no joint flexibility (V1) for wave direction at 270° wrt N

Analysis V2

Repeating Analysis V1 but applying the slightly higher Section 17.0 ultimate strength loads for the diagonal North East direction, gives the results shown in Figures 8 to 10 and Table 7. The same distributed plasticity model (Element Type 31) is adopted with no account of joint flexibility and with rigid foundation assumptions.

It can be seen from Figure 9 that the ultimate capacity is limited by regions of plasticity due to portal action in the legs. As the peak load is approached, plasticity develops in the thicker leg sections in the bay below failure sites 1 and 2. The distribution of the loads through all four bracing frames ensures that the members and joints are not overstressed.

At the point the analysis is stopped the Section 17.0 ultimate strength load has been sustained with a greater margin than for V1.



Ls17u - Sec. 17.0 Ultimate Strength Load	2830 Kips
Reference Level Load	2420 Kips
Load Level at First Member Failure	4057 Kips
Ultimate Capacity (Ru)	4740 Kips
Reserve Strength Ratio	1.959
Platform Failure Mode	Jacket Legs

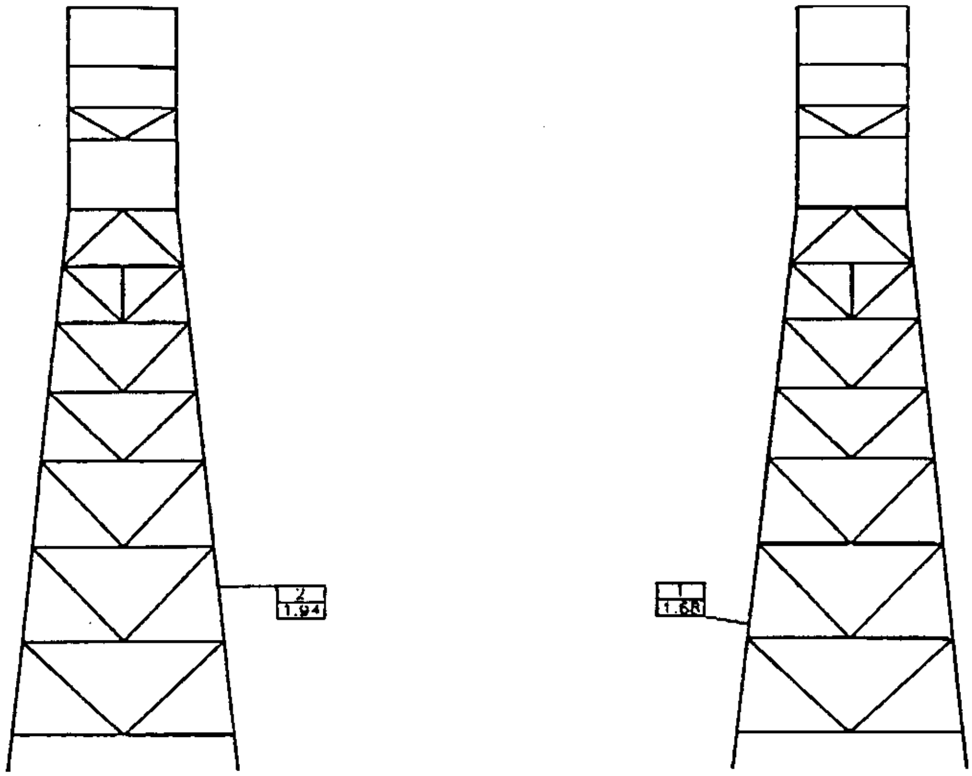
Figure 8: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V2) for wave approach direction at 225° wrt N

Lateral load level at yield of the first leg section is 4057 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.295	1754		
3	0.592	3509		
4	0.700	4057	1	Yield in compression/bend.
5	0.767	4349		
6	0.795	4453		
7	0.838	4601		
8	0.868	4708	2	Yield in tension/bending
9	0.882	4740		

* See Figure 9 indicating location and sequence of component failures

Table 7: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V2) for wave approach direction at 225° wrt N



Platform North frame

Platform South frame

Figure 9: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case
with no joint flexibility (V2) for wave approach direction at 225° wrt N

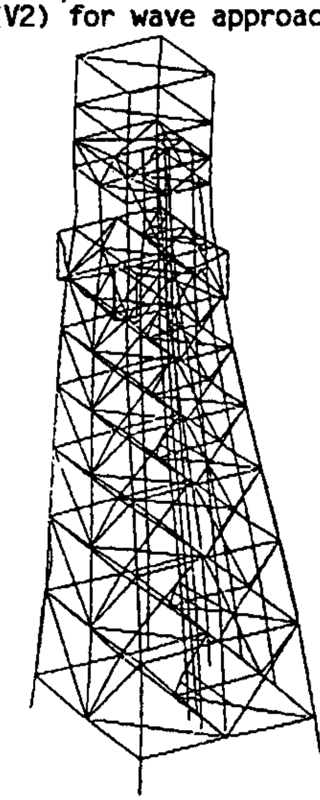


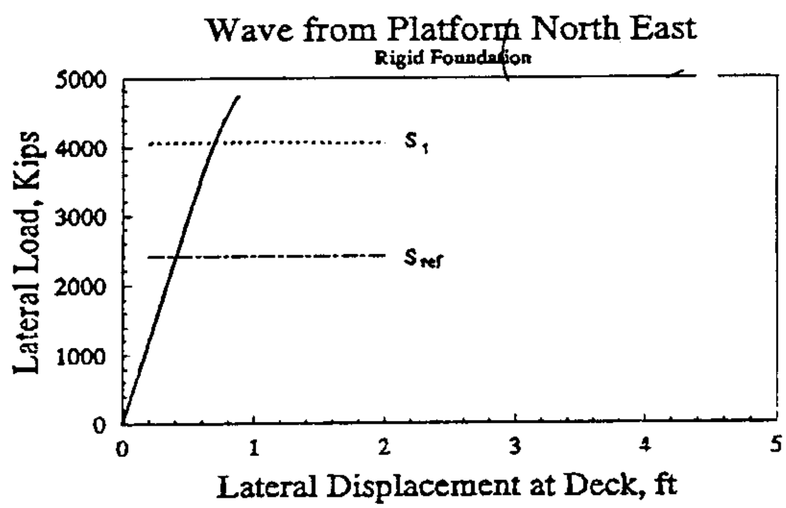
Figure 10: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case
with no joint flexibility (V2) for wave direction at 225° wrt N (mag. x 20)

Analysis V3

Repeating Analysis V1 but applying the lower Section 17.0 ultimate strength loads for the orthogonal Northerly direction gives the results shown in Figures 11 to 13 and Table 8. The same distributed plasticity model is adopted with no account of joint flexibility and with rigid foundation assumptions. An alternative plastic hinge analysis was also conducted confirming the peak capacity.

Despite the different wave/current combination and the different disposition of the conductors with respect to the incident loads, the symmetry of the primary jacket framing ensures that the load level at first member failure is very similar to the orthogonal case V1 (compare Figures 4 and 11). However the Section 17.0 ultimate strength load for this direction is significantly lower and at the point the K-braced members yield and buckle together under combined axial forces and bending (see Table 8 and Figure 12) a much higher reserve strength has been achieved. In an assessment, based on the findings for V1, it can be said that by inspection further scrutiny of the tubular joint responses is not required as this V3 analysis corresponds to a less critical direction.

Table 8 (viewed in conjunction with Figure 12) indicates that developing plasticity in both the tension and compression K-bracing between levels -126ft and -71ft has driven the automatic introduction of nonlinear distributed plasticity elements along the entire member lengths. At the peak load all components shed load together giving a rapid unloading. This contrasts with the V1 case where the stiffness gradually reduced with a sequence of component failures.



Ls17u - Sec. 17.0 Ultimate Strength Load	2830 Kips
Reference Level Load	2420 Kips
Load Level at First Member Failure	4057 Kips
Ultimate Capacity (Ru)	4740 Kips
Reserve Strength Ratio	1.959
Platform Failure Mode	Jacket Legs

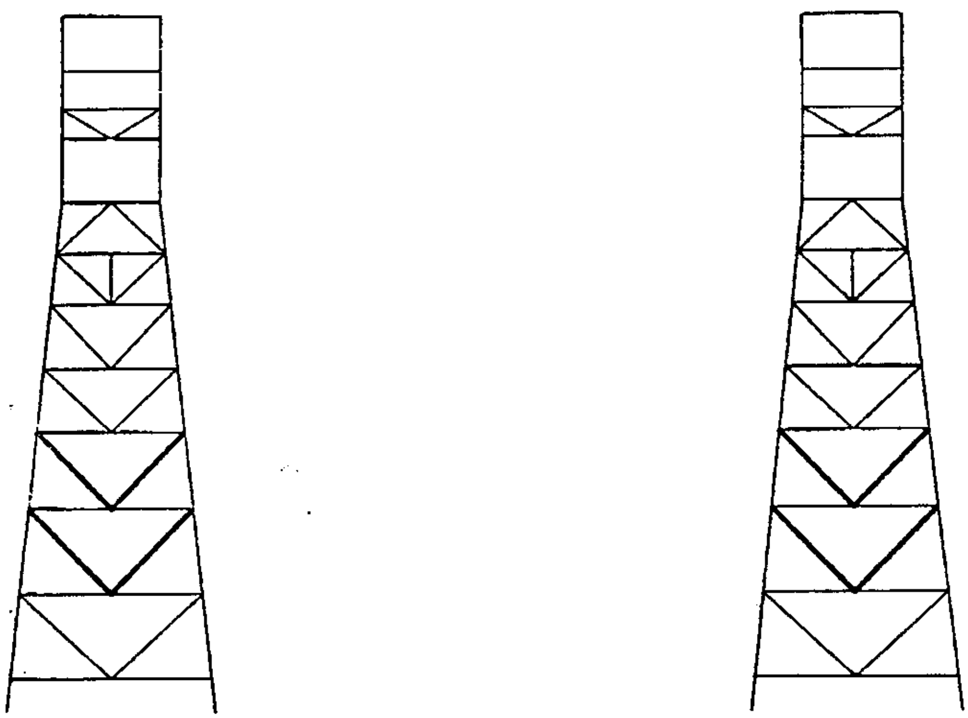
Figure 11: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N

Lateral load level at failure of the bracing is 4196 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.260	1590		
3	0.590	3613		
4	0.689	4196	1-4 / 5-8	Yield/buckle
5	0.754	3835		
6	0.800	3732		
7	1.000	3464		
8	1.213	3326		
9	1.508	3224		
10	1.745	3214		

* See Figure 12 indicating location of component failures

Table 8: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N



Platform West frame   Platform East frame

Figure 12: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N

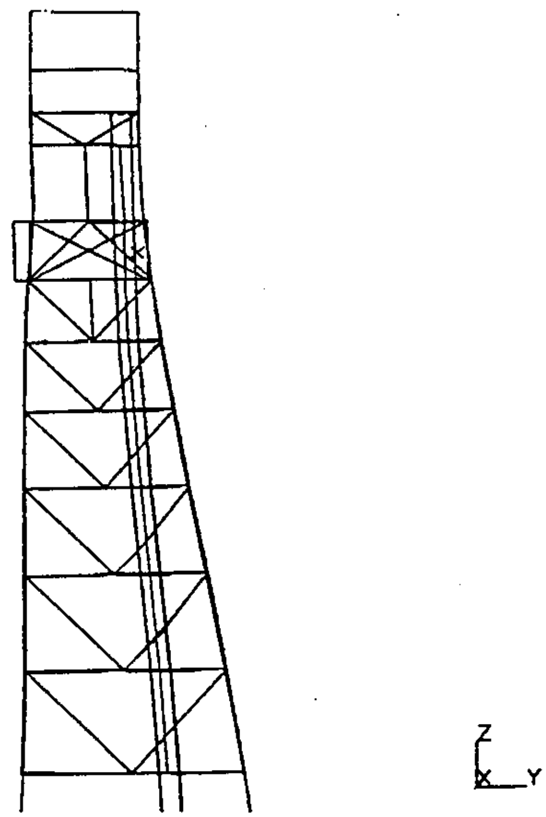


Figure 13: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case with no joint flexibility (V3) for wave direction at 180° wrt N (mag. x 20)

Analysis V4

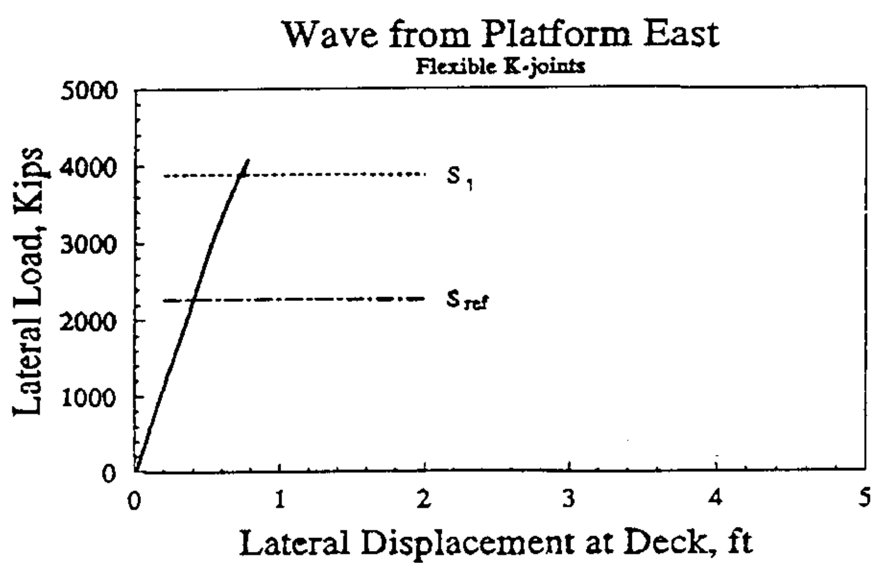
In analysis V4 the base V1 model is adopted but account is taken of the flexibility of the primary K joints between levels -126ft and -27ft as these were shown to be highly utilised in the V1 analysis. The results are presented in Figures 14 to 16 and Table 9.

The code checks to API RP 2A 20th Edition with safety factors removed demonstrated in Analysis V1 that the joint loading would reach the code lower bound before the mean strength of the members (as simulated in the analysis) was achieved. Two problems are faced: Section 17.0 is directed at a best estimate representation of system capacity with all bias removed: the joint deformations will be considerable prior to the attainment of the limiting capacity and may be limited by subsequent fracture. A direct application of the API RP 2A lower bound as a limiting capacity fails to address both these points.

In analysis V4 the nonlinear axial and in-plane and out-of-plane bending resistances at the joints is modelled with decoupled springs with 3 to 5 part linear representations of the flexibility and softening responses, as appropriate. SAFJAC analyses base the limiting capacities on the equations published in OTH 89 308 as a mean fit to the data underlying the HSE Guidance Notes strength equations. The source material for the same data forms the basis of parametric stiffness equations used within SAFJAC. Elastic flexibility limits and peak deformations are determined on the basis of tubular joint parameters, β, γ, ζ, θ etc. for ready implementation in the analysis.

The HSE equations afford significantly higher capacity to β=1.0 K joints than API and for this reason joint failures in Analysis V4 do not have a drastic influence on the response as was suggested by the RP 2A code check in Analysis V1. The HSE K joint capacity has been verified in recent experimental tests on components and frames in the JIP R&D project. In analysis V4 it is found that the joint flexibility contributes to differences in the load distribution such that a very slightly higher peak load is attained. The RSR is 1.80 in comparison with 1.78 for Analysis V1. The influence of joint flexibility is also seen in the less stiff global response as the peak load is reached. The first capacity limit is reached for a North frame K joint. However. the failure precipitating global load shedding is a member. as shown in Figure 15. This demonstrates the ability of SAFJAC to evaluate load interaction effects leading to collapse of the member separately from joint conditions. This ensures that the critical component is dictated by the analysis and load interaction effects within the structure and not the user. Allowance is made for fracture and rapid load shedding at the K joint replicating physical evidence for similar joints exposed to Hurricane Andrew and in experimental ultimate strength tests. This dictates the sharp load reduction beyond the peak global load.

Figure 15 shows the regions of member plasticity and the location of tubular joints where the capacity has been exceeded. In Figure 16 bowing of the members can be seen.



Ls17u - Sec. 17.0 Ultimate Strength Load	2702 Kips
Reference Level Load	2268 Kips
Load Level at First Joint Failure	3880 Kips
Ultimate Capacity (Ru)	4077 Kips
Reserve Strength Ratio	1.797
Platform Failure Mode	Jacket

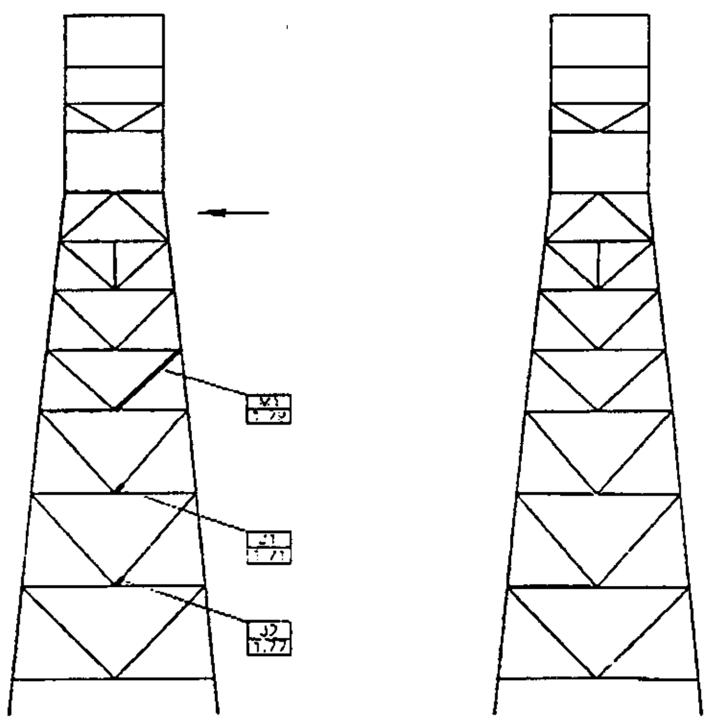
Figure 14: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N

Lateral load level at failure of the first K joint is 3880 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.395	2214		
3	0.536	3017		
4	0.715	3880	J1	K joint
5	0.752	3989		
6	0.770	4053	J2	K joint
7	0.777	4077	M1	Buckle
8	0.742	3860		
9	0.751	3890		

* See Figure 15 indicating location and sequence of component failures

Table 9: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N



Platform North frame   Platform South frame

Figure 15: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N

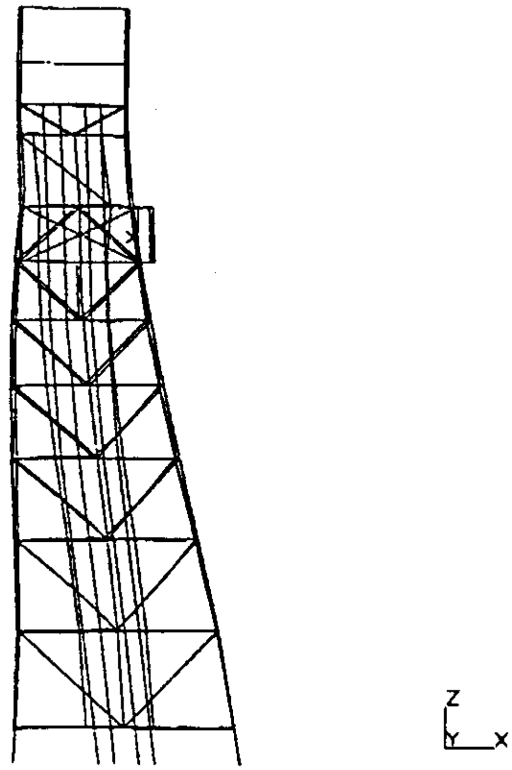


Figure 16: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case with flexible joints (V4) for wave direction at 270° wrt N (mag. x 20)

The series of voluntary analyses and comparison with the required case. demonstrate the influence of modelling assumptions. Figure 17 compares three results fro the 270° wave attack direction. For the specific installation. the foundation compliance has a significant influence on the response. Within this structure. the joints have little influence on the ultimate strength once proper account is taken of their mean capacity but the mode of failure and post-peak response are significantly different.

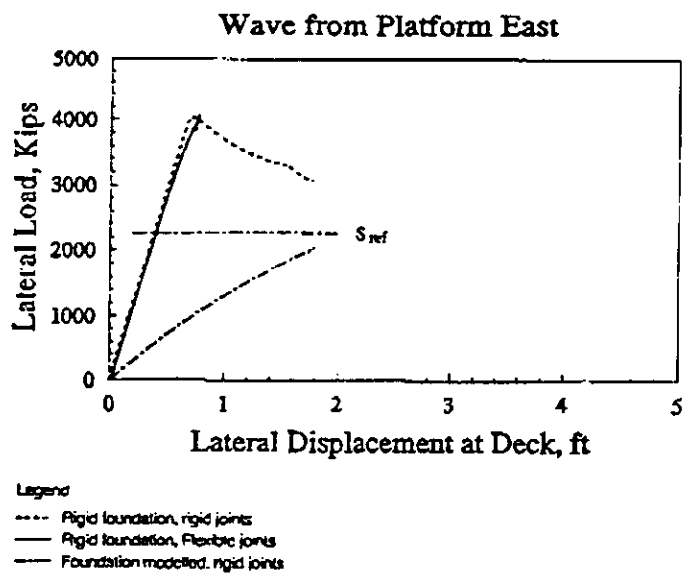


Figure 17: Ultimate Strength Analyses
Load displacement results for wave direction at 270° wrt N (mag. x 20)

PART B: REVIEW AND FEEDBACK TO THE API TG 92-5

In this section comments are made on the use of the draft Section 17.0 (April 29 version) as applied to the Gulf of Mexico benchmark structure.

Software

In 17.7.2b and 17.7.3b it is recommended that the clauses read "software developed *and validated* for that purpose".

Conductors

In the final clause of the Commentary, C17.7.3c (g), it is required that the gap between jacket and conductor be modelled. Clearly this is aimed at realism, however there is uncertainty in the initial position of the conductor in the slot. For this reason the added complexity may not necessarily lead to an improved representation of the system behaviour. Perhaps it need not routinely be modelled but if the criteria are only just met this and other factors such as initial member out-of straightness etc should be recommended for inclusion in a sensitivity study.

Ultimate strength modelling

Section 17.7.3c provides instructions on element grouping and this is expanded significantly in the commentary. It is questioned whether the level of guidance in the guideline itself is helpful. It is suggested that the clause should reiterate the intention to use best estimate properties to model components (as stated explicitly for foundations) and indicate that, if required, further guidance on the grouping of similar elements for modelling purposes is contained in the commentary.

The discussion regarding the modelling of structural members in the commentary appears to be written with the concepts of an "INTRA" type analysis in view. Other programs which have been developed and validated for ultimate strength analysis have automatic facilities to accommodate large deflection beam column action including the effects of end fixity without requiring the user to select specific K factors or element types before performing an analysis. It is also unnecessary to scrutinise working stress analysis results to establish which element types should be selected for each location "based on the dominant stresses". These software packages make the single step to ultimate strength check increasingly viable from economic and time standpoints.

Perhaps a more general approach would be to state that the modelling should properly account for beam column effects, the potential onset of plasticity, and the effect of frame restraints on buckling capacity etc. This generality leaves the analyst better able to interpret the guideline and less likely to give inadequate consideration to factors which may cursorily be disregard as irrelevant.

Deck loading

The presentation of deck loading in the commentary C17.6.2 could be open to different interpretation. For example wave loads on the net silhouette area are readily distributed equally to decks above and below. In reality structural members might share the load top to bottom whereas loads incident on equipment/structure standing on the deck will pass loads to the lower level almost exclusively. Should the net area modelling be associated with the nett deck area for attracting loads rather than the between deck silhouette. Alternatively the proposed procedure may be adequate but should perhaps be flagged for further investigation in a sensitivity study should the margin beyond the required ultimate strength be small.

Typographical

17.3.1c "platform *is* not"

17.5.2 "environmental" - remove space and hyphen

17.6.2b "Section 17.6.2a.2" ? These is no 17.6.2a.5

17.7.3 " to ensure adequacy"

17.7.3c "deformation"

PARTICIPANT "L"

1.0 Summary

The platform was initially analyzed elastically to determine its strength or weakness to withstand the various load levels it is to be analyzed for, including the ultimate strength level storm. The results of this analysis indicated that the structure is substantially weak, and very limited in its capacity to resist the ultimate strength level storm. Specifically, its ultimate capacity is less than 2/3 of that required for the ultimate strength storm, and first yield is less than 1/2 of that.

The analysis for ultimate capacity of the structure was carried out using the pushover method and the StruCAD*3D computer program. Because of the low resistance level of the structure it became evident that a simplified non-linear analysis for member and joints will provide an efficient yet effective analysis approach. In this simplified approach, when member failure is reached, the member stiffness is set to zero (same as removing the member) and its end forces and moments are applied back to the structural frame throughout the subsequent load step increases. The soil non-linearity is fully accounted for.

Member failure is determined by the unity ratio computed according to API RP 2A with all safety factors removed. P-delta effect is included. Joint check is also performed at each step to ensure joints have not failed before member failure. In case of a major joint failure, the corresponding member load is limited to that load.

A detailed computer model was generated. Piles and conductors below the mudline were modelled as piles with non-linear soil curves. Wind areas of the deck, wave areas of the boat landing and barge bumpers and inundated deck areas were modelled.

As mentioned before, a preliminary linear analysis had indicated that the platform is unable to take even 70% of the ultimate level wave load due to pile failures, even if member failures are disregarded. Single pile analysis for simple tension and compression loads had confirmed that at this load level the pile axial loads due to overturning effect and gravity exceed the punch-through and pull-out capacity of the soil. In the structural analysis, redistribution of pile loads through the jacket bottom framing was considered. At ultimate storm load level, several members had also shown unity ratios larger than 1.0. Therefore, the wave load was further scaled down till the first member just fails and then subsequent simplified "non-linear" analyses performed.

The results are summarized below:

Wave Direction 1: 180 degrees w.r.t x-axis

Load at First member failure, S_1	=	1102 kips
Reference Level Load, S_{ref} (20 th edition)	=	2235 kips
Ultimate wave base shear	=	2755 kips
Ultimate Capacity, R_u	=	1804 kips
Reserve Strength Ratio, $RSR = R_u/S_{ref}$	=	0.807
Platform Failure Mode		Jacket and piles

Wave Direction 2: 225 degrees w.r.t x-axis

Load at First member failure, S_1	=	1315 kips
Reference Level Load, S_{ref} (20 th edition)	=	2419 kips
Ultimate wave base shear	=	2922 kips
Ultimate Capacity, $R_u = R_u/S_{ref}$	=	1811 kips
Reserve Strength Ratio, RSR	=	0.748
Platform Failure Mode		Jacket and piles

A.1 Environmental Criteria

The "benchmark platform" is classified under Manned Evacuated / Significant Environmental Impact. The Metocean criteria for the platform analysis for this condition has been obtained from Table 17.6.2-1 and Figure 17.6.2-2a of the API RP 2A - WSD, Section 17 (Draft).

The x-axis of our model runs towards platform East. Initial review had shown that wave approach directions of 180 degrees and 225 degrees are the most critical, therefore, these directions have been analyzed. The criteria for each direction are summarized below:

Data for Environmental Loads

Seastate Criteria used in analysis:	Number of approach direction 2

Direction: 1

Wave direction w.r.t platform North	90 degrees (180 degrees w.r.t x-axis)
Wave height	64.6 ft
Wave period	13.5 sec
Current profile	2.185 knots (3.69 ft/sec) - uniform from mudline to El +1')
Storm surge	4 ft
Wind speed @ 10 m above msl	85 knots (143.6 ft/sec)
Magnitude of wave load in deck	127 kips
Total base shear	2755 kips

Direction: 2

Wave direction w.r.t platform North	135 degrees (225 degrees w.r.t x-axis)
Wave height	68 ft
Wave period	13.5 sec
Current profile	2.3 knots (3.88 ft/sec) - uniform from mudline to El +1')
Storm surge	4 ft
Wind speed @ 10 m above msl	85 knots (143.6 ft/sec)
Magnitude of wave load in deck	140 kips
Total base shear	2922 kips

Stream Function wave theory has been used for the wave kinematics. Current stretching and Doppler shift effects and wave kinematic factor of 0.88 have been included in accordance with API RP 2A, 20th edition.

Approach used to determine wave/current deck forces

For the ultimate level wave height, the wave crest level including tide is determined as 50 ft. The subcellar deck being at 33 ft and lower deck at 43 ft, the deck height criteria is not met. Therefore, there would be wave loading on the deck for the ultimate condition. For the purpose of determining wave load on the deck we have used PMB specified wave area and calculated the drag forces assuming a C_d value of 2.0 for 180 degrees and 1.5 for 225 degrees wave direction in accordance with API RP 2A Section 17. The wave areas given by PMB have been subdivided in the vertical direction for more accurate estimate.

A.2 3-D Model Generation

The computer model includes a detailed modelling of the jacket members, piles, conductors and deck. The pile is represented as ungrouted through the use of "wish-bone" members. The piles and conductors below the mudline have been modelled up to full depth and soil has been represented by non-linear p-y, t-z and q-z curves. These curves have been developed based on API RP 2A recommendations. The risers have been modelled to generate wave loads only. Cans of jacket legs and stubs of K-braces have been modelled. Boat landing and barge bumper wave areas have been included. For wind load calculations API wind profile has been used along with appropriate wind areas.

Data for Member Capacity Estimation

Member yield strength	42 ksi
Member capacity estimate:	
Braces	API interaction curves with all safety factors removed. K factors have been used as per API RP 2A.
Legs/Piles	API interaction curves with all safety factors removed. Piles inside jacket legs were assumed braced against column buckling.
Joint capacity estimate	API strength checks have been used, with all safety factors removed.
Soil spring (p-y, t-z, q-z)	Based on PMB specified properties for the soil the curves have been generated using API RP 2A, 20th edition recommendations

A.3 Software Description

The structure was analyzed using the push-over analysis method and StruCAD*3D software. A simplified non-linear procedure was used sequentially updating the member stiffness to account for member failure. The procedure is outlined below:

(1) Initial linear analysis for the full ultimate wave load had demonstrated that there would be several member failures. Therefore, the wave load has been scaled down and combined with full gravity load to determine at what level the first member failure occurs.

(2) Member failure has been defined when the unity check according to API code check formulae is equal to 1.0 with all safety factors removed.

(3) When a member fails, it is replaced by its internal forces and moments at the two ends, and its stiffness is reduced to zero. This is equivalent to assuming that at initial member failure, its load carrying capacity is held constant. This assumption is appropriate for yielding type of brace failures anticipated for the structure.

(4) In the subsequent analysis load step, the proportion of wave load is increased slightly and the modified structure with the stiffness of the failed member removed, is analyzed. The load is increased till further member failure occurs.

(5) P-delta effect has been included. A joint check has been performed at each stage.

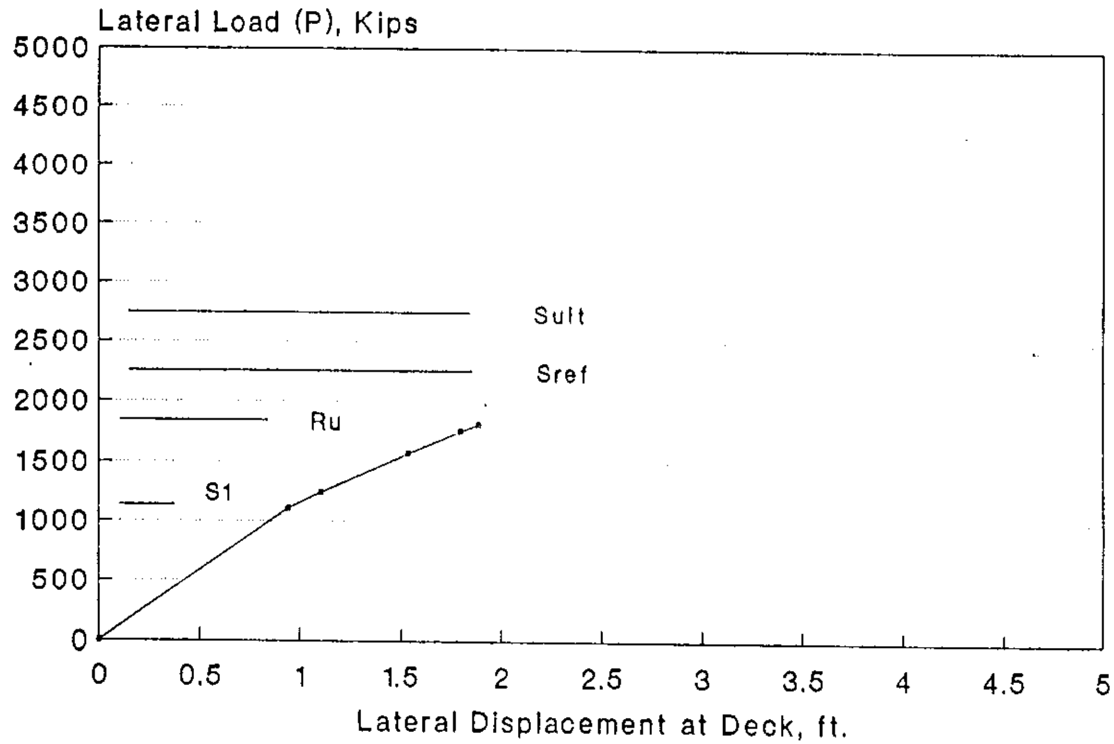
(6) This process is continued till the maximum loading for the structure is reached.

A.4 Ultimate Strength Analysis

Platform members fail at relatively low load levels. First member failure starts at the mudline horizontal framing level. There were a few joint failures at even lower load levels. The members associated with these joint failures were not primary members. Therefore, these were considered numerical failures in view of the presence of mudmats and conductor guide area stiffening which were ignored in the model. Only joint failures which occurred in the major vertical framing were investigated. The load deflection data are shown in the following figures and tables.

The final collapse occurs as a result of foundation failure for both the wave directions. At these load levels, the compression piles yield at the pile head. For the diagonal wave direction, the soil axial capacity is also exceeded.

Ultimate Strength Analysis
Benchmark Platform



Load at First member failure, S_1	=	1102 kips
Ultimate wave base shear, S_{ult}	=	2755 kips
Reference Level Load, S_{ref}	=	2235 kips
Ultimate Capacity, R_u	=	1804 kips
Reserve Strength Ratio, RSR	=	0.807
Platform Failure Mode:		Jacket, Pile

Figure 1: Load - Displacement Results for 180 degrees w.r.t x-axis

Direction 1 : Analysis with Foundation

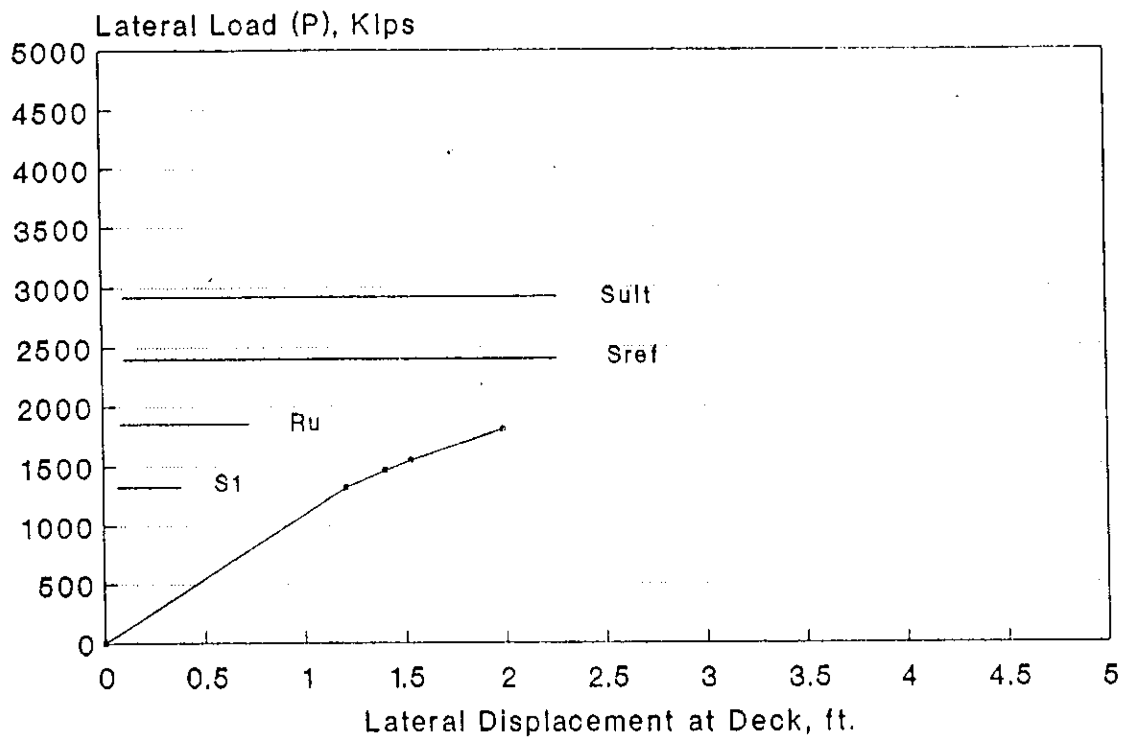
Lateral load level for first member failure 1102 Kips

Load Step	Lateral Displacement at Deck Level (+43') ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.94	1102	505*	Yielding	
2	1.10	1239	503, 504	Yielding	
3	1.53	1570	139, 140 (joint)	Joint failure	
4	1.79	1749	147,148 (joint);500,506	Joint failure, mem. yielc	
5	1.88	1804	132 (joint); 499	Joint failure, mem. yielc	Foundation failure
6					beyond this load
7					
8					
9					
10					

* Member number

Table 1: Ultimate Strength Analysis for 180 degrees to x−axis

Ultimate Strength Analysis
Benchmark Platform



Load at First member failure, S_1	=	1315 kips
Ultimate wave base shear, S_{ult}	=	2922 kips
Reference Level Load, S_{ref}	=	2419 kips
Ultimate Capacity, R_u	=	1811 kips
Reserve Strength Ratio, RSR	=	0.748
Platform Failure Mode		Jacket, Piles

Figure 2: Load - Displacement Results for 225 degrees w.r.t x-axis

Direction 2 : Analysis with Foundation

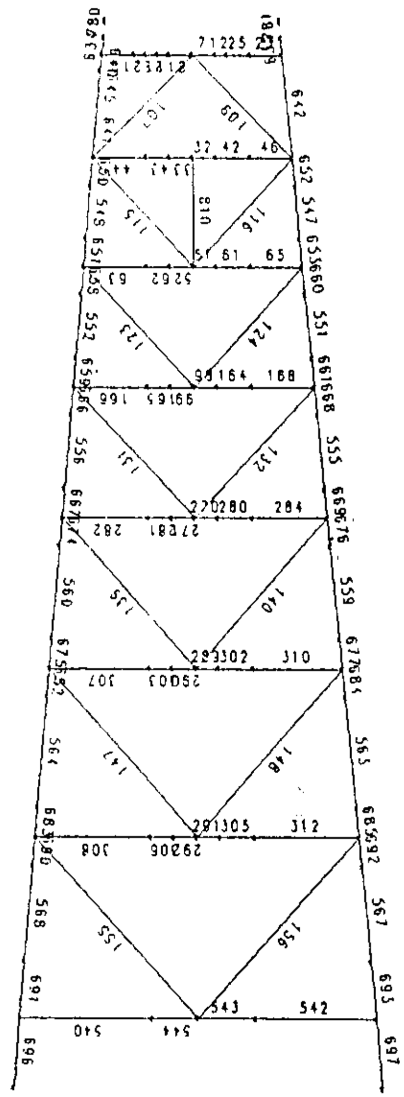
Lateral load level for first member failure 1315 Kips

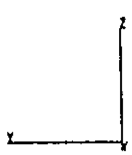
Load Step	Lateral Displacement at Deck Level (+43') ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	1.20	1315	503*	Yielding	
2	1.40	1461	505	Yielding	
3	1.53	1548	504	Yielding	
4	1.99	1811	500	Yielding	Foundation failure
5					beyond this load
6					
7					
8					
9					
10					

* Member number

Table 2: Ultimate Strength Analysis for 225 degrees to x-axis

Element numbers at North face





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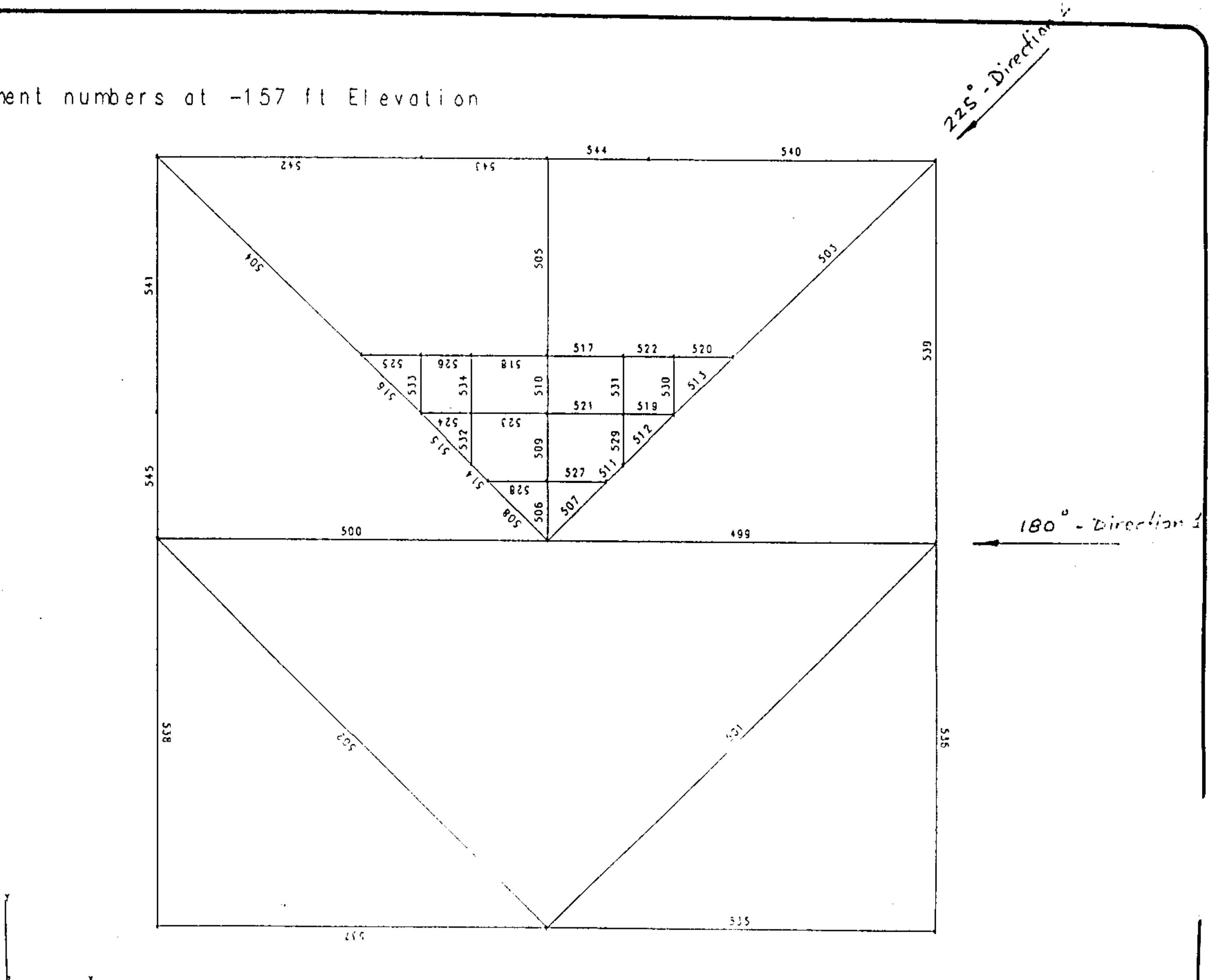
Element numbers at -157 ft Elevation



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PART B: Review and Feedback to the API TG 92-5

B.1 Ultimate Strength Analysis Criteria - Environmental

A philosophical background for Section 17 should be added as introduction (subsection 17.1) explaining what we are trying to do, so that user can appreciate why different wave heights (as compared to 100 year waves, 20th edition) have to be used for design level or ultimate level checks as well as for different exposure categories.

PARTICIPANT "M"

1.0 Summary

The computer programs used for the Benchmark analyses, are modules of the SESAM program package in combination with USFOS.
SESAM programs are mainly developed by Det Norske Veritas Sesam, and USFOS is developed by SINTEF.

The program modules from SESAM have been utilised for generation of the beam-model (PREFRAME) and for the wave/current/buoyancy calculation/distribution (WAJAC). Wave and current loads are calculated from the 100-year environmental conditions, using Morison's equation and Stream function with order 9 wave theory.

USFOS has been utilised for the non-linear pushover analyses and for presentation of computer model and results.

Further description of the software used, is presented in section A-3.

Global jacket coordinate system has positive x-axis in true SE direction, positive y-axis in true NE direction and positive z-axis upwards. The axis is placed at mudline in centre of the jacket.

A total of 3 environmental directions have been analysed. Directions/degrees with respect to true North (clockwise) are 245°, 290° and 335°. Both the pile foundation case (non-linear soil springs) and the fixed base case (soil/structure fixed to the sea bed) have been reported.

Results

Foundation Case:

Environmental direction	Ultimate Capacity	Reserve Strength Ratio **
245°	2563.60 Kips	0.99 *
290°	2445.60 Kips	0.75 *
335°	2613.00 Kips	1.00

* Pile punch through the soil, leading to collapse of the platform.
** Reserve Strength Ratio (RSR) is the capacity beyond the 100 year env. load.

Table 1.0-1 Results - Foundation case

Fixed Base Case:

Environmental direction	Ultimate Capacity	Reserve Strength Ratio **
245°	3568.10 Kips	1.38
290°	3581.90 Kips	1.10
335°	3512.00 Kips	1.34

** Reserve Strength Ratio (RSR) is the capacity beyond the 100 year env. load.

Table 1.0-2 Results - Fixed Base case

A-1 Environmental Criteria

The loads of 3 wave directions have been established. Design wave heights and current profile are selected for the specific platform location. The wave period is modified to include Doppler effects. Stream function with order 9 wave theory have been selected. Linear current stretching method is used. Drag coefficients of 0.65/1.05 and mass coefficients of 1.6/1.2 have been applied for smooth members (above El. +1'- 0") and rough members (from mud line to El. +1'- 0") respectively. Marine growth is set to 1.5", and applied up to El. +1'-0".

Seastate Criteria used in the different pushover analyses are tabulated below.

Env. Dir.	Wave Height	Wave Period	Current Profile	Storm Surge	Wind Speed **
245° *	56.70 ft.	13.85 sec.	3.502 ft./sec.	3.5 ft.	135.025 ft./sec.
290° *	63.00 ft.	13.85 sec.	3.542 ft./sec.	3.5 ft.	135.025 ft./sec.
335° *	59.85 ft.	13.85 sec.	3.472 ft./sec.	3.5 ft.	135.025 ft./sec.

* Direction with respect to Platform true North, clockwise

** Wind speed @ 10 m above msl

Table A-1-1 Seastate Criteria

The wave/current deck forces caused by horizontal runs of risers and a 4' x 8' x 4' sump tank, are calculated by hand according to API-RP2A-WSD 20th Edition - Draft guidelines.
It should be noted that the drag coefficient (Cd) corresponding to moderately equipped deck has been selected. Drag coefficient equal to 2.0 (end-on and broadside) is applied for env. dir. 245°/335°, and Cd equal to 1.5 (diagonal) for env. dir. 290°.

Following loads have been applied on the simplified deck model:

Env. Dir.	Wave/Current deck forces
245°	65.4 Kips
290°	128.2 Kips
335°	107.8 Kips

Table A-1-2 Wave/current deck forces

A-2 3-D Model Generation

The 3-D computer model has been generated using the SESAM PREFRAME pre-processor. The computer model consists of 362 nodal points and 598 elements. All elements have been simulated by use of 2-noded beam elements with 6 fundamental degrees of freedom at each node. Both geometry and materials includes non-linear properties. The structure is modelled with one finite element per physical member (see also A-3 software description). Jacket geometry and section properties used, reflect the platform structural drawings presented in the Benchmark Basis Document.
Wave/current deck forces has been calculated by hand and applied to the computer model. Deck loads and topside wind loads are applied as concentrated loads on the deck nodes. Gravity, wave, current and buoyancy loads are applied as distributed forces along the jacket members. Weight of upper- and lower deck structure have been met by manipulating the density of the simplified deck model.

Non-linear response behaviour between soil/structure has been simulated by non-linear springs (foundation case).
Following characteristic values have been applied for the 36" piles (non-linear characteristics for the conductors can be provided upon request):

Pile Axial Load Kips	Pile Axial Deflection Inches	Remark
250	0.03782	
500	0.10291	
1000	0.34351	
2000	1.10293	
3000	2.18696	
3250	2.35362	Threshold value - pile capacity

Table A-2-1 Axial load/deflection for the 36" piles

Pile Lateral Load Kips	Pile Lateral Deflection Inches
100	0.64178
140	1.14297
180	1.78059
220	2.70106
250	3.69587
275	4.7884
350	9.9969
400	15.55285
500	33.8543

Table A-2-2 Lateral load/deflection for the 36" piles

Pile Moment Kips-inches	Pile Rotation Radians
10000	0.002472
18000	0.004661
24000	0.006466
32000	0.009027
40000	0.011706
52000	0.015983
60000	0.019045

Table A-2-3 Moment/rotation for the 36" piles

Following simplifications have been done on the computer model:

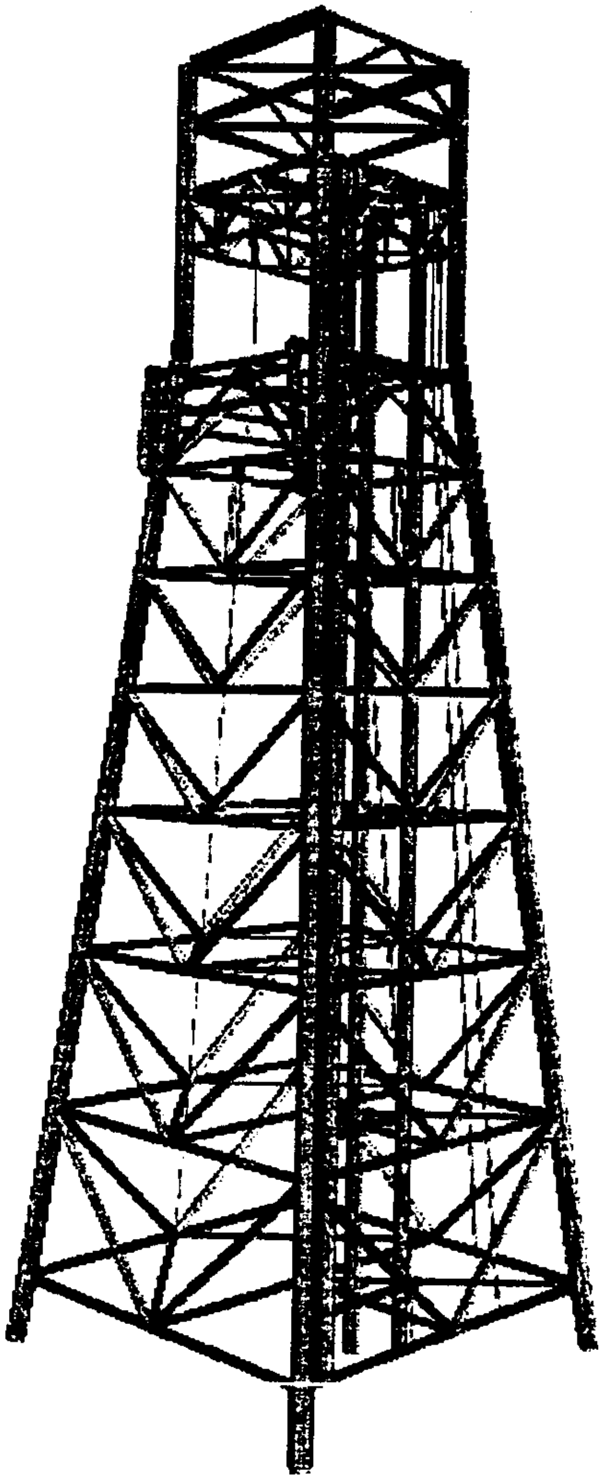
- Stubs/cans are not included in the model.
- Risers (8 off) which spans from El. +43.0' to mud line, are merged into 4 distinct groups. These groups are located taking individual cross-section area into account. Real dimensions are used. The risers are modelled for free axial movement, but constrained to follow the lateral displacement of the jacket.
- The guides for the conductors are simplified, no gap between guide and conductors are modelled.
- Boat landing and bumpers located on platform east and south side are simplified. Hydrodynamic loads are calculated and given hydrodynamic diameters specifying the relevant area.
- Material yield strength is assumed as 42 ksi for all elements involved.
- The model representing plated deck structures are simulated by cross bracing's. Following formula has been applied:

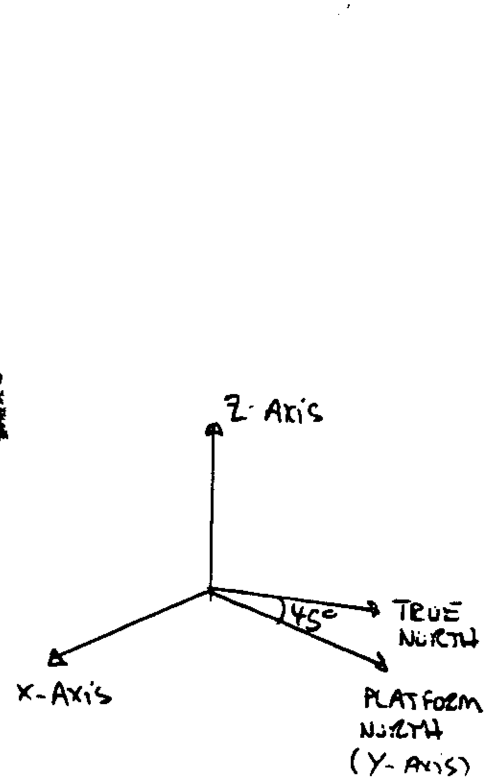
  $A_d = (1+k^2)^{1.5} * h*t / 4 (1+ v) * k$; where:

A_d	-	bracer cross-section
h x l	-	main dimensions of the plated deck
t	-	plate thickness
k	-	l/h
v	-	Poison's ratio (0.3)

An overall view of the computer model is presented over leaf.

Fig. A-2-1 Computer Model - Benchmark Jacket:





A-3 Software description

USFOS has been utilised for the non-linear pushover analyses. The program follows an updated La Grange (incremental-iterative) procedure. It uses a non-linear Green strain formulation with the von Karman approximation. The stiffness formulation of USFOS is derived from potential energy consideration.
The plasticity model is formulated in stress resultant space, based on the bounding surface concept. Two interaction surfaces are used, one yield surface representing first fibre yield, and one bounding surface representing the full, plastic capacity of the cross section. When the cross section is loaded, the force point travels through the elastic region until it reaches the yield surface. When further loading takes place, the yield surface travels with the force point, such that the force point stays on the yield surface.
The plastic behaviour of a member is defined by the (non-linear) elastic stiffness, the strain hardening and a parameter describing the elastic-plastic transition for each cross sectional force component (axial force, bending moment etc.)

The program uses a 12 - d.o.f. geometrically non-linear beam-column element, with a plastic hinge formulation to model material non-linearity. It uses only one finite element per physical element of the structure, i.e to use the same finite element discretization as in linear, elastic analyses. In this context, it should be noted that USFOS utilise the SESAM interface file format.

The USFOS non-linear analysis follows the following basic procedure:

- Load is applied in steps
- Nodal coordinates are updated after each step
- The structure stiffness is assembled at each load step. Element stiffness are then calculated from the updated geometry
- At every load step, each element is checked to see whether the forces exceed the plastic capacity of the cross section. If such an event occurs, the load step is scaled to make the forces comply "exactly" with the yield condition.
- A plastic hinge is inserted when the element forces have reach the yield surface. The hinge is removed if the element later is unloaded and becomes elastic.
- The load step is reversed (the load is reduced) if global instability is detected.

A-4 Ultimate Strength Analysis

A total of 3 environmental directions have been analysed. Both pile foundation case (non-linear soil springs) and fixed base case (soil/structure fixed to the sea bed) are reported.

Tabulated values of lateral loads and deflections representing the different plots are presented. It should be noted that only a few characteristic values are selected from the computer analysis as basis for the history plots.

Failure mechanism:

- Foundation base:

The axial capacity for the 36" piles is 3250 Kips. When this threshold value is exceeded, the piles will punch through the soil. The platform lateral deflections increase significantly leading to collapse of the platform. This mechanism occurs for all environmental directions analysed.
See Computer plot A-4-1P enclosed for visualisation of the failure mechanism, wave direction as indicated.

- Fixed base:

First member failure and member failure leading to platform collapse, seems to be located between elevation -27.0' and -8.0' (Ref. Computer plot A-4-2P enclosed). The leg elements located in this area will reach yield as "first member failure". Since the pile is located inside the leg , collapse of the leg element will depend on the pile behaviour. Leg elements which are axial compressed (pile tensioned), have to obtain significant strains when further increase of the lateral load take place. Regarding the system of pile and leg, the sum of axial forces is tension. Therefore, it is assumed that no buckling of the leg element will take place before the elements yield capacity is reached. After yield capacity is reached, the element is assumed to be capable to "hold" the actual compression load until the magnitude of the lateral load lead to a complete platform collapse. Leg elements and K-bracers located in the actual area (between el. -27.0'/-8.0') and leg elements located in area between el. -48.0'/-27.0' seem to form a possible platform failure mechanism (Ref. Computer plot A-4-2P enclosed).

Platform failure mechanism for the fixed base case indicated above, occurs for all environmental directions analysed.

Tabulated values and diagrams for the environmental directions analysed, are presented on the next pages. Deflected shapes of the platform when significant non-linear events have formed, are showed for typical failure modes enclosed (computer plots A-4-1P and A-4-2P).

Environmental direction 245° (with respect to true North), fixed and foundation base case (plot):

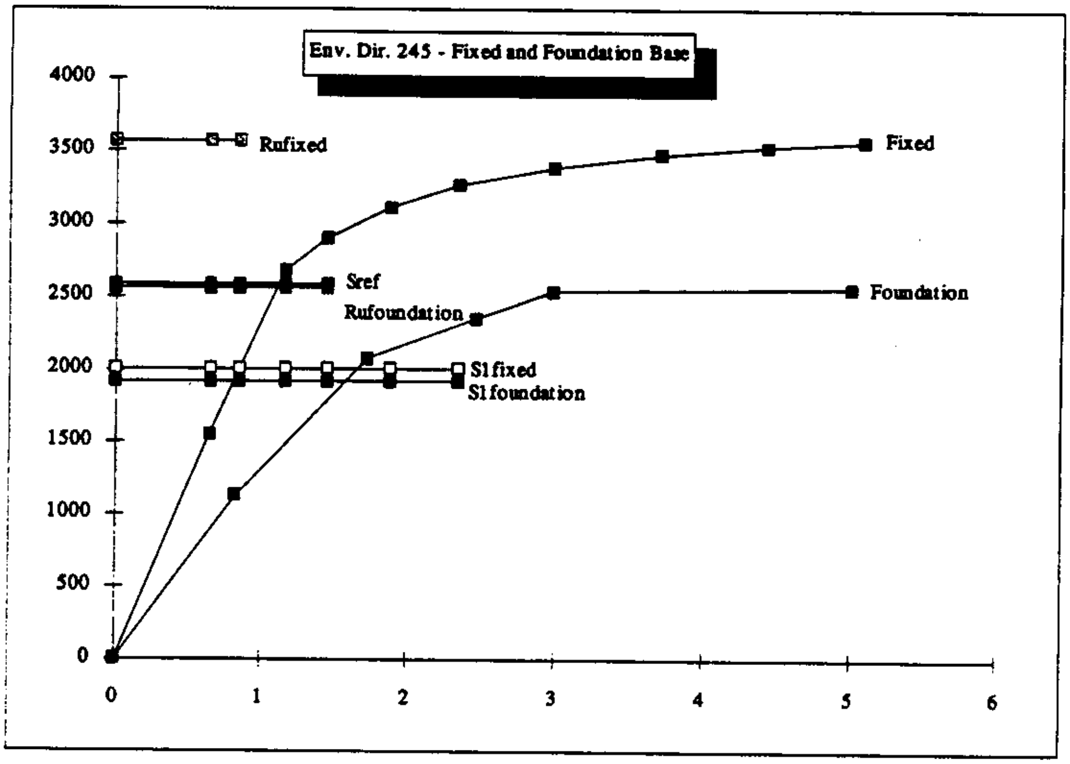


Fig. A-4-1 Load/displacement - pushover analyses, 245°. (Load in Kips upwards, displ. in feet horizontal)

Abbreviations:

Sref	:	Reference Level Load corresponding to the 100-year seastate criteria per API RP 2A-WSD, 20th Edition.
S1foundation, S1fixed	:	Load level at First Member Failure (foundation base and fixed base respectively)
Rufoundation, Rufixed	:	Ultimate Capacity (foundation base and fixed base respectively)

	Foundation Base	Fixed base
Reference Level Load (Sref)	2589.50 Kips	2589.50 Kips
Load Level at First Member Failure (S1foundation/S1fixed)	1921.40 Kips	2006.70 Kips
Ultimate Capacity (Rufoundation/Rufixed)	2563.60 Kips	3568.10 Kips
Reserve Strength Ratio (RSR) **	0.99 *	1.38
Platform Failure Mode	Soil	Jacket

* Piles punch through the soil

** Reserve Strength Ratio (RSR) is capacity beyond the 100 year env. load.

Environmental direction 245° (with respect to true North), fixed and foundation base case (tab. values):

Tabulated values corresponding to the plot on the previous page are presented below.

Foundation Base:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Foundation Base Case	Remarks
0	0	
0.832	1125.4	
1.59	1921.4	First Member Failure
1.73	2076.8	
2.46	2351.3	
2.99	2540.3	
5.0	2563.6	Ultimate Capacity

Table A-4-1 Tabulated Load/displacement 245° - Foundation Base

Fixed Base Case:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Fixed Base Case	Remarks
0.0	0.0	
0.65	1553.6	
0.85	2006.7	First Member Failure
1.01	2395.1	
1.16	2677.4	
1.45	2897.4	
1.89	3115.0	
2.34	3265.1	
2.99	3384.3	
3.71	3472.3	
4.43	3531.9	
5.09	3568.1	Ultimate Capacity

Table A-4-2 Tabulated Load/displacement 245° - Fixed Base

Environmental direction 290° (with respect to true North), fixed and foundation base case (plot):

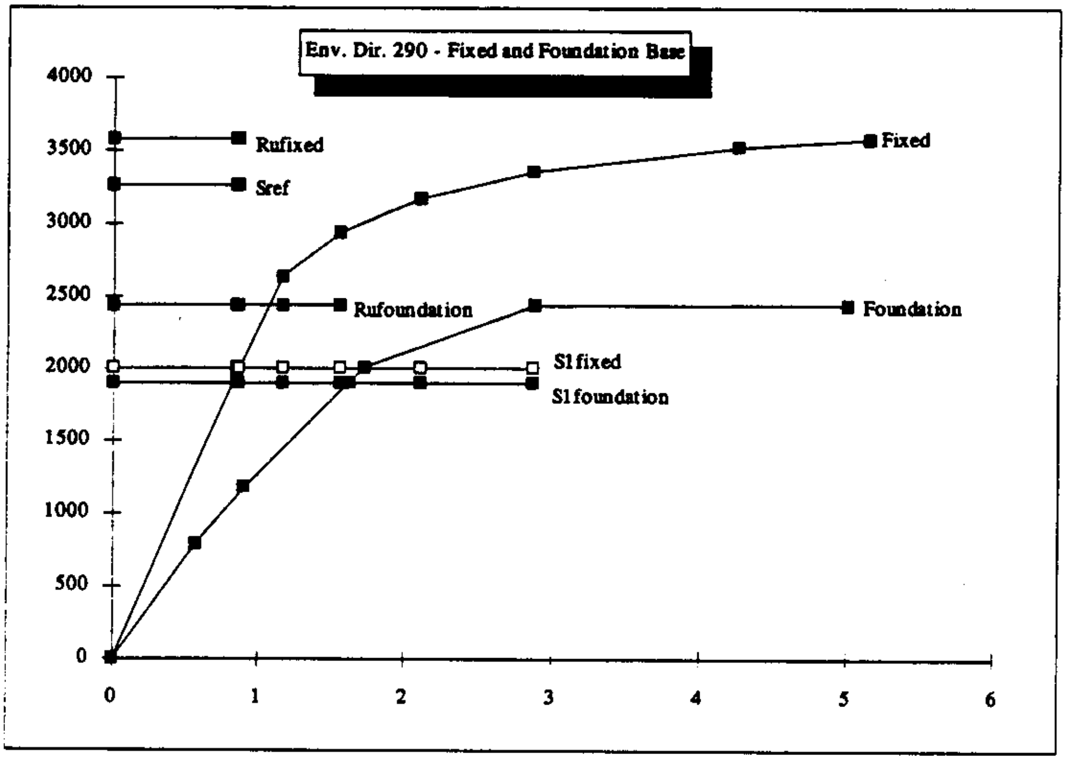


Fig. A-4-2 Load/displacement - pushover analyses, 290°. (Load in Kips upwards, displ. in feet horizontal)

Abbreviations:

Sref	:	Reference Level Load corresponding to the 100-year seastate criteria per API RP 2A-WSD, 20th Edition.
S1foundation, S1fixed	:	Load level at First Member Failure (foundation base and fixed base respectively)
Rufoundation, Rufixed	:	Ultimate Capacity (foundation base and fixed base respectively)

	Foundation Base	Fixed base
Reference Level Load (Sref)	3265.18 Kips	3265.18 Kips
Load Level at First Member Failure (S1foundation/S1fixed)	1900.30 Kips	2004.80 Kips
Ultimate Capacity (Rufoundation/Rufixed)	2445.60 Kips	3581.90 Kips
Reserve Strength Ratio (RSR) **	0.75 *	1.10
Platform Failure Mode	Soil	Jacket

* Piles punch through the soil

** Reserve Strength Ratio (RSR) is capacity beyond the 100 year env. load.

Environmental direction 290° (with respect to true North), fixed and foundation base case (tab. values):

Tabulated values corresponding to the plot on the previous page are presented below:

Foundation Base:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Foundation Base Case	Remarks
0	0	
0.575	790.2	
0.91	1178.7	
1.62	1900.3	First Member Failure
1.73	2011.4	
2.88	2439.1	
3.38	2445.6	Ultimate Capacity

Table A-4-3 Tabulated Load/displacement 290° - Foundation Base

Fixed Base Case:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Fixed Base Case	Remarks
0	0	
0.85	1959.1	
0.87	2004.8	First Member Failure
1.17	2644.8	
1.56	2935.4	
2.11	3173.8	
2.87	3356.6	
4.26	3523.1	
5.15	3581.9	Ultimate Capacity

Table A-4-4 Tabulated Load/displacement 290° - Fixed Base

Environmental direction 335° (with respect to true North), fixed and foundation base case (plot):

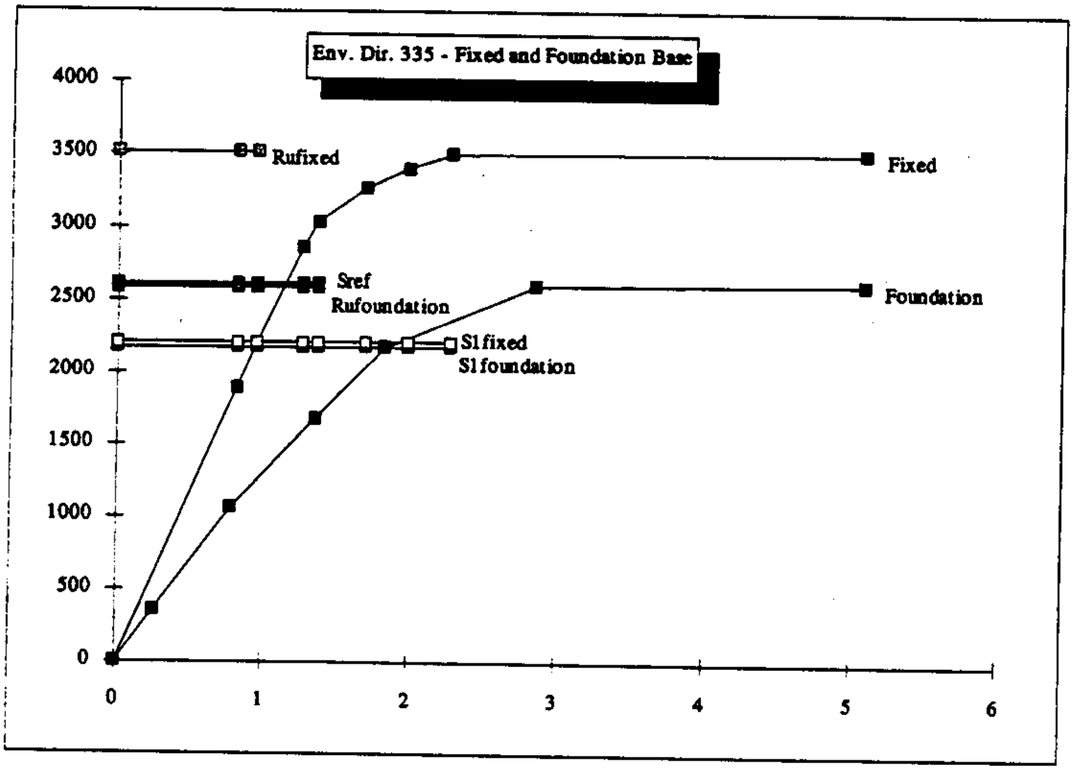


Fig. A-4-3 Load/displacement - pushover analyses, 335°. (Load in Kips upwards, displ. in feet horizontal)

Abbreviations:

Sref	:	Reference Level Load corresponding to the 100-year seastate criteria per API RP 2A-WSD, 20th Edition.
S1foundation, S1fixed	:	Load level at First Member Failure (foundation base and fixed base respectively)
Rufoundation, Rufixed	:	Ultimate Capacity (foundation base and fixed base respectively)

	Foundation Base	Fixed base
Reference Level Load (Sref)	2613.00 Kips	2613.00 Kips
Load Level at First Member Failure (S1foundation/S1fixed)	2174.00 Kips	2210.60 Kips
Ultimate Capacity (Rufoundation/Rufixed)	2613.00 Kips	3512.00 Kips
Reserve Strength Ratio (RSR) **	1.0	1.34
Platform Failure Mode	Soil	Jacket

** Reserve Strength Ratio (RSR) is capacity beyond the 100 year env. load.

Environmental direction 335° (with respect to true North), fixed and foundation base case (tab. values):

Tabulated values corresponding to the plot on the previous page are presented below:

Foundation Base:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Foundation Base Case	Remarks
0	0	
0.26	358.0	
0.79	1063.5	
1.36	1680.0	
1.84	2174.0	First Member Failure
2.86	2597.3	
5.0	2613.0	Ultimate Capacity

Table A-4-5 Tabulated Load/displacement 335° - Foundation Base

Fixed Base Case:

Lateral Displacement at Deck Level (+43') South East Leg - feet	Lateral Load Kips Fixed Base Case	Remarks
0	0	
0.82	1886.6	
0.96	2210.6	First Member Failure
1.27	2866.5	
1.37	3033.7	
1.70	3263.6	
1.99	3402.1	
2.28	3498.8	
2.34	3512.0	Ultimate Capacity

Table A-4-6 Tabulated Load/displacement 335° - Fixed Base

Computer plot A-4-1P Deflected shape of the platform when significant non-linear events have formed. Foundation base (non-linear soil springs). Soil failure:

PARTICIPANT "A" RESUBMITTAL

PARTICIPANT "A"
PLATFORM RE-ASSESSMENT

Table of Contents

1.0 SUMMARY

Participant "A" performed a static Push Over analysis for a "Benchmark" platform located in the Gulf of Mexico. The provided soil information and the API RP 2A formulations were used to determine foundation response to the applied loads on the piles. The drawings provided and information made available on platform functional loads and member properties were used in the development of the computer model.

The developed computer model (see Section 3) was used in determining the applied environmental loads on the platform. Specific discussion of the benchmark analysis is presented in the following sections.

This document summarizes the results of a re-assessment of the benchmark platform. The initial assessment, submitted in August 1994, contained a number of input and data interpretation errors which produced erroneous analysis results. The more significant errors are attributable to misinterpretation of the API RP 2A Section 17 Criteria and in the input of the given soil p-y and t-z data. Additional discussion on these errors is presented in Appendix A.

Further discussion with other participants reveals that assumed pile capacity varies among the participants. It would be desirable to correlate the benchmark analysis results with assumed pile capacity to ascertain if this assumption significantly affects the benchmark analysis or if the pile capacity has even been accounted for in the analysis. We are aware of one participant who has a higher assumed pile capacity but a lower reported platform ultimate capacity than has Participant "A." If this is true, then normalizing with respect to pile-soil capacity would provide a wider range of analysis results.

2. ENVIRONMENTAL CRITERIA

Environmental criteria varied with storm analysis direction. The three directions chosen for the analysis were:

Platform Loading	Site Loading	
South	Southwest	(Direction 1)
Southwest	West	(Direction 2)
West	Northwest	(Direction 3)

Wave, current and wind data for the platform site were determined based on both Draft Section 17 of API RP 2A and the 20th edition of API RP 2A. Current directionality was accounted for in the combined wave and current acting on the platform.

The seastate criteria determined for this benchmark reanalysis are summarized as follows:

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
NORTH				
Wave Height, H(ft)	45.0	60.7[1]	56.7	Note 1: Includes 0.90 height correction factor
Wave Period, T (sec)	11.3	12.5	13.0	
Current				
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.2	2.3	1.2	
Velocity (fps)	2.0	3.9	2.1	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	
WEST				
Wave Height, H(ft)	45.0	67.4	63.0	
Wave Period, T (sec)	11.3	12.5	13.0	
Current				
Blockage Factor	0.85	0.85	0.85	
Velocity (knots)	1.2	2.3	2.1	
Velocity (fps)	2.0	3.9	3.5	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	
NORTHWEST				
Wave Height, H(ft)	45.0	64.1[2]	59.9	Note 2: Includes 0.95 height correction factor
Wave Period, T (sec)	11.3	12.5	13.0	
Current				
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.2	2.3	1.8	
Velocity (fps)	2.0	3.9	2.9	
Wind (1 hr at 10m)				
velocity (knots)	55	70	80	
velocity (fps)	63	80.5	92	

3. 3-D MODEL GENERATION

A three-dimensional model of the platform was developed modeling all platform legs and vertical frame diagonal and horizontal plan braces. Member properties were as per the provided drawings. Deck plan framing was not specifically modeled, but represented as equivalent cross-braced framing to represent the stiffness and load transfer characteristics of the actual structure.

4. SOFTWARE DESCRIPTION

The benchmark analysis was performed using in-house developed computer program ASADS. ASADS is a three-dimensional linear and nonlinear static and dynamic analysis computer code with specific pre- and post-processor program enhancements for marine and offshore engineering applications.

Joint coordinates consist of a joint number or name and coordinates in a Cartesian (x-y-z) coordinate system. Beam-type member and other finite element incidences are defined to connect the joints as required. The model topology may be further refined by specifying joint and member releases and joint eccentricities at member ends.

Member properties amy be input directly or specified directly as a function of the member cross-sectional shape. Tubular, conical transition, AISC shapes, plate girder and box girder shapes are presently available and may be input as single segment prismatic or multiple-segmented variable sections.

The model may be further enhanced using ASADS' node mensuration and detailing facilities. The node mensuration facility determines joint chord and brace members, included angles between all intersecting members and prints warning messages relevant to the structure connectivity and sizing. This facility is particularly useful for checking the validity of the modelled structure and associated design.

Specified lead conditions consist of joint and/or member loads or joint displacements and/or rotations. Member loads may consist of concentrated, uniform or linearly varying member loads at or over any portion of the member.

Automatic load generation is available for modelled dead, buoyancy, ballast, wind, wave and current loads. Direct dynamic inertial load generation for periodic wave and current loads is also available as a single-line command once a dynamic modal analysis has been performed.

ASADS' present analysis capabilities consist of:

- standard static linear stiffness analysis
- linear static stiffness analysis with non-linear pile-soil interaction
- iterative static "pushover" analysis
- dynamic modal analysis via subspace iteration
- dynamic spectral response analysis

Linear and nonlinear static stiffness analyses are performed via a one-line command with automatic bandwidth optimization included.

Iterative static pushover analyses were used for this benchmark analysis. This analysis capability is provided via a PC-ANSR module incorporated directly into ASADS. The program source code was enhanced to allow processing of larger models typical of offshore platforms and to properly account for P-M interaction of tubular cross-sections as nonlinear beam-column elements.

This ASADS database is translated to a PC-ANSR compatible format along with additional user-specified parameters to describe the non-linear properties of the analysis model. ASADS offers the capability to directly transfer the nonlinear properties to the PC-ANSR module for previously specified tubular, wide flange, plate or box girder, or pile-soil members. Presently available nonlinear elements available include linear-elastic truss, nonlinear truss, linear-elastic beam-column, nonlinear beam-column, gap and nonlinear brace elements. PC-ANSR is made available to the public through the National Information Service for Earthquake Engineering - NISEE/Computer Applications, University of California, Berkeley.

5. SUMMARY OF APPLIED LOADS

Functional loads acting on the platform are as follows:

- Deck equipment and variables weight: 1,425 kips
- Deck and jacket self-weight (steel): 1,431 kips
- Jacket net buoyancy: 1,209 kips

The wind, wave and current loads generated along the three platform axes are as follows:

DESCRIPTION	SOUTHWEST (Direction 1)	WEST (Direction 2)	NORTHWEST (Direction 3)
DESIGN LEVEL			
Base Shear (Kips)	1108	1328	1084
O.M. (Kip-feet)	149,000	172,000	146,000
ULTIMATE STRENGTH			
Base Shear (Kips)	2100	2500	2280
O.M. (Kip-feet)	271,000	334,000	298,000
20TH EDITION API RP 2A			
Base Shear (Kips)	1440	2040	1780
O.M. (Kip-feet)	185,000	269,000	232,000

6. ULTIMATE STRENGTH ANALYSIS

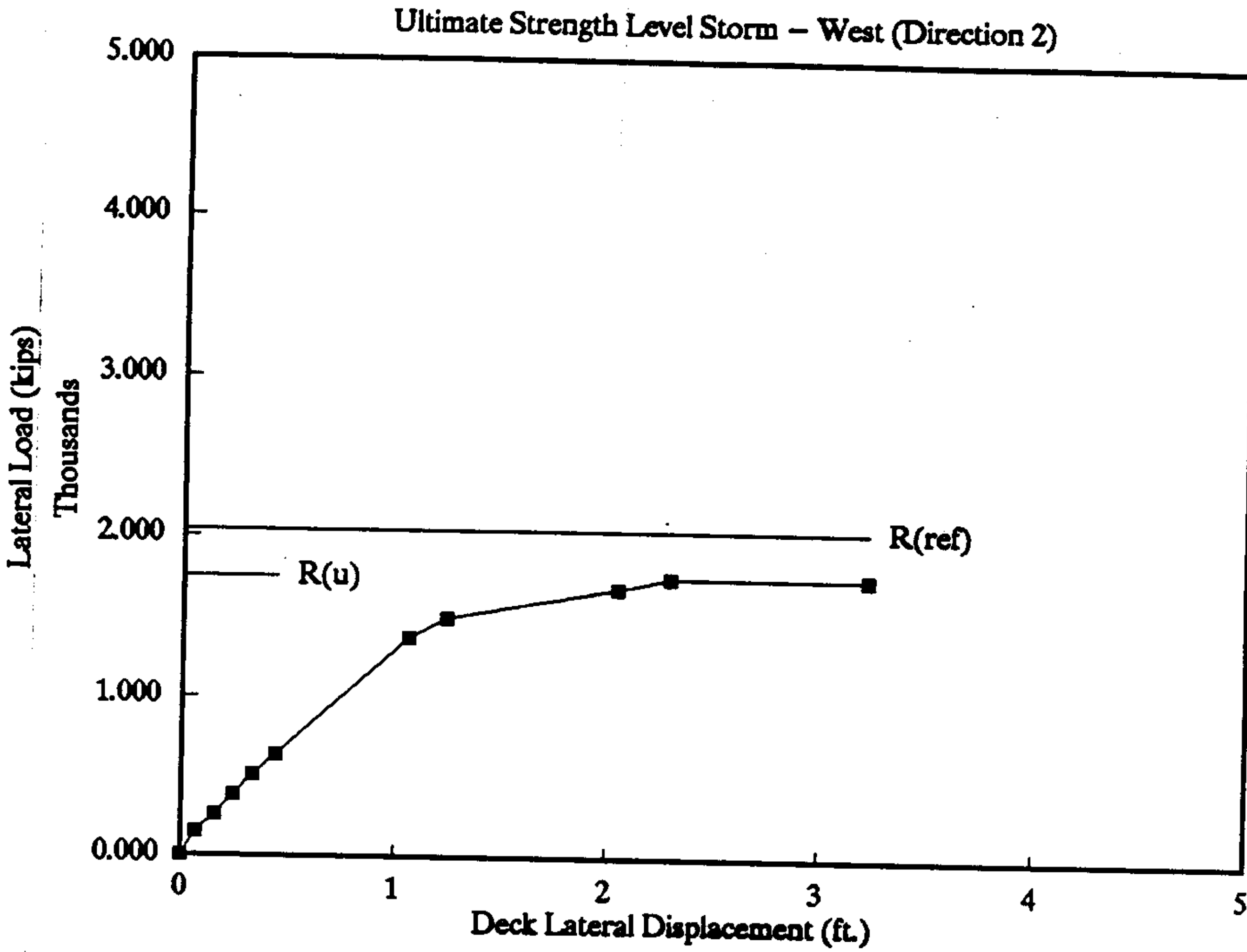
An ASADS pushover reanalysis was performed for two storm wave directions; platform west and southwest. An ultimate strength storm load condition that maximized platform base shear was generated for each storm direction and incrementally applied to the model. The applied load increment was 5 percent of the ultimate strength level loading.

All tubular members were modeled as nonlinear beam-column members. Compression capacity was determined based on member slenderness ratios. The API recommendations for "K" were assumed, i.e., 1.0 for leg members and 0.8 for braces. The moment capacity of the member was reduced as a function of the present axial load per [Chen and Han, 1985], namely:

$$M / M_{cr} = \cos[(pi / 2) (P / P_{cr})]$$

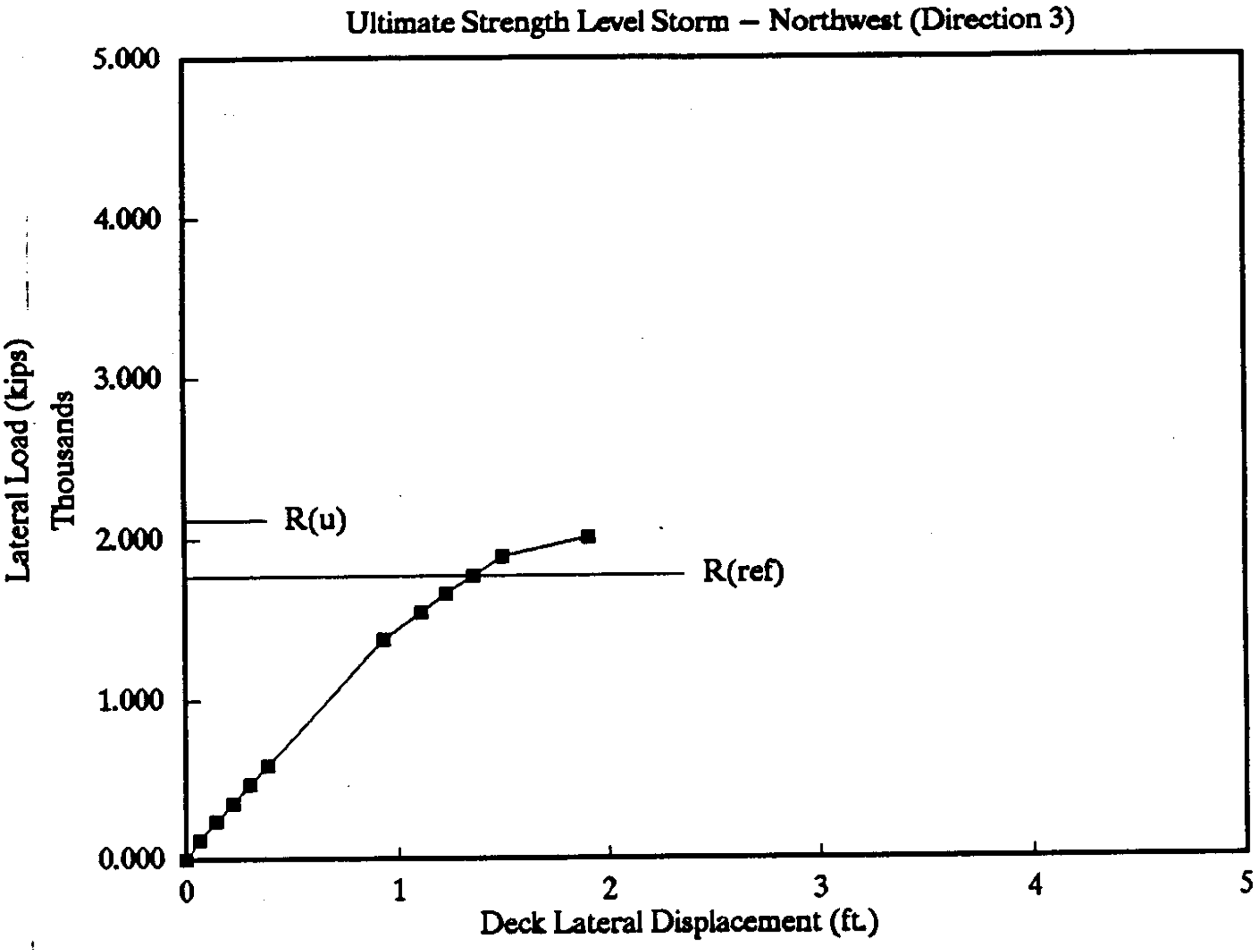
The piles within the legs were modeled to provide the correct stiffness and the capacity of the components to resist axial and bending loads. Although the piles at jacket/pile interface have the geometric and material properties for strain hardening, this characteristic was not incorporated into this study.

Load-displacement plots for the west and northwest (platform southwest and west) storm directions follow.



Reference Level Load, (R_{ref})	2,035 kips
Design Level Load (DLL)	1,328 kips
Ultimate Strength Level Load (USL)	2,495 kips
Ultimate Capacity (R_u)	1,750 kips
Reserve Strength Ratio (RSR)	0.86
Platform Failure Mode: Jacket, Pile, Soils, etc.	Soil Compression Capacity

Figure A.6-1
Ultimate Strength Level Load-Displacement Results
Platform West Direction (Direction 2)



Reference Level Load, (R_{ref})	1,775 kips
Design Level Load (DLL)	1,084 kips
Ultimate Strength Level Load (USL)	2,278 kips
Ultimate Capacity (R_u)	2,120 kips
Reserve Strength Ratio (RSR)	1.19
Platform Failure Mode: Jacket, Pile, Soils, etc.	Compression Piles

Figure A.6-2
Ultimate Strength Level Load-Displacement Results
Platform Northwest Direction (Direction 3)

APPENDIX A

DATA RESUBMITTAL - PARTICIPANT "A"

JIP - TRIAL APPLICATION OF API RP 2A SECTION 17
BENCHMARK ANALYSIS

<u>DATA RESUBMITTAL - PARTICIPANT "A"</u>

1. SUMMARY

On August 10, 1994 Participant "A" submitted its initial benchmark analysis results to the JIP. This initial analysis effort was an optional activity to the Trials JIP and was performed at Participant's expense on a part-time basis. Participant's overall workload was very high during the time of the benchmark analysis, increasing as the due date for the initial submittal drew near. Therefore, Participant did not have the time or resources to devote a full quality effort to the benchmark analysis and a number of errors and misinterpretations were made. This resubmittal of Participant's analysis is intended to detail the errors and misinterpretations of the initial submittal and revise the affected results.

Participant "A" reviewed its initial analysis and performed the benchmark analysis a second time for two of the three analyzed storm directions. The review showed that "gross errors" were made in the initial work, most relating to input into the analysis model. The second analysis results conform to those of the other Participant's.

2. INITIAL ANALYSIS ERRORS

Several errors were made by Participant in the initial analysis. The significant errors were as follows:

- API RP 2A Section 17 Criteria

    - The "insignificant environmental impact" environmental criterion was used in the initial analysis rather than "significant environmental impact" criterion as requested in the benchmark platform description. This was an inadvertent oversight on Participant's part and is the reason for the low wave height and differing storm directions used in the ultimate load generation relative to the other participants. As a result of this error and the relative difficulty in interpreting the Section 17 and 20th Edition metocean criteria recipes, the software has now been enhanced to allow output of metocean criteria as a function of platform location, water depth, environmental criteria and life safety criteria. While this enhancement will not ensure elimination of the error made in Participant's initial analysis, it will provide for consistent analysis criteria from one analysis to another in the future.

■ Soil Modeling

- The pile-soil axial load (t-z) data was incorrectly input into the software creating an almost rigid axial stiffness of the pile-soil element. This is the major cause of the high initial stiffness of Participant "A" in the draft report load-displacement response curves. There was also a slight error in the development of the p-y curves per the API RP 2A procedures.

■ Modeling Assumptions

- Our initial report stated that the piles were assumed grouted to the legs. This statement was in error. In fact, this assumption was not incorporated into the initial nor the second analysis. The increased stiffness of our initially reported platform response was due to the soil modeling error discussed above.

3. REVISED BENCHMARK ANALYSIS RESULTS

The draft report tables have been marked-up to reflect revisions applicable to Participant "A." The marked-up tables follow.

4. OTHER SOLICITED RESULTS

A request soliciting response on various topics was issued. Our results, where available, are as follows:

■ <u>Base case - with pile/soil effect considered</u>

- Wave in deck loading estimates? Wave crest elevation used?

The attached marked-up tables present our results for wave-in-deck load. The tabulated results represent wave load on the platform structure above Elevation 16'-0. These in-deck wave loads and the associated maximum wave crest elevations are as follows:

	Wave-In-Deck Load (kips)	Max. Wave Crest Elev. (feet)
Direction 1	45	+38.2
Direction 2	236	+42.7
Direction 3	126	+36.5

- How were the conductors modeled? Were conductors modelled to contribute to foundation capacity?

The conductors were modeled as 48 or 30 inch diameter tubulars, 5/8 inch thick. The full length from conductor tip to conductor top was modeled. Soil response on the conductors was included; thus, the conductors were modeled to contribute to foundation capacity.

YY - 270° 225 60.70 3.83 1,943 156 2,099 56.70 2.08 1,386 45 1431

Table 3-1: Comparison of Metocean Parameters and Loads: Direction 1 (225 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	In-Line Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	In-Line Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	45	[illegible]	3.00	?	?	[illegible]	56.70	?	?	?	[illegible]
B	225	[illegible]	[illegible]	1,821	319	2,140	[illegible]	[illegible]	1,545	125	1,670
C	225	61.20	2.40	1,895	100	1,995	56.70	2.19	1,570	90	1,660
D	225	55.00	?	1,352	38	1,390	52.50	[illegible]	1,273	17	1,290
E	225	61.20	[illegible]	2,732	48	[illegible]	-	-	-	-	-
F	-	-	-	-	-	-	-	-	-	-	-
G	225	60.30	?	?	?	2,004	56.70	?	?	?	2,174
H	225	61.20	3.11	?	?	2,038	56.70	2.84	?	?	1,669
I	225	56.23	?	?	?	1,992	[illegible]	?	?	?	1,600
J	225	55.00	2.06	?	?	?	?	?	?	?	1,948
K	225	55.83	3.76	1,840	40	1,880	?	?	?	?	1,590
L	-	-	-	-	-	-	-	-	-	-	-
M	245	-	-	-	-	-	56.70	3.50	2,525	65	[illegible]
Mean		58.31	2.88	1,928	109	2,007	56.04	2.77	1,728	74	1,725
St. Dev.		4.04	0.78	500	120	494	2.45	0.79	548	45	432
COV		0.07	0.27	0.26	1.10	0.25	0.04	0.28	0.32	0.61	0.25

Notes:
#1: "?" indicates that information is not available.
#2: The gray boxes identify the lower and upper values for columns.

225° 270 67.45 3.88 2,105 390 2,495 63.00 3.49 1791 236 2027

Table 3-2: Comparison of Metocean Parameters and Loads: Direction 2 (270 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads: Wave Ht., H-17 (ft.)	Section 17: Current, U-17 (ft/sec)	Section 17: Wave Load on Jacket (kips)	Section 17: Wave-in-Deck Load (kips)	Section 17: Total Base Shear, S-17 (kips)	RP2A, 20th Edition, Metocean Parameters and Loads: Wave Ht., H-20 (ft.)	RP2A: Current, U-20 (ft/sec)	RP2A: Wave Load on Jacket (kips)	RP2A: Wave-in-Deck Load (kips)	RP2A: Total Base Shear, S-20 (kips)
A	360	[illegible]	[illegible]	?	?	[illegible]	61.10	?	?	?	1,150
B	270	68.00	3.36	2,241	406	2,727	63.00	3.07	1,952	118	2,070
C	270	68.00	3.86	2,500	355	2,935	63.00	3.52	2,065	165	2,230
D	270	64.00	?	1,492	98	1,590	[illegible]	3.54	1,247	63	1,310
E	270	68.00	3.86	3,152	78	3,230	-	-	-	-	-
F	270	68.00	?	1,870	364	2,234	?	?	?	?	1,713
G	270	67.00	3.88	3,330	99	[illegible]	63.00	3.54	?	?	2,810
H	270	68.00	3.31	?	?	2,992	63.00	[illegible]	?	?	2,318
I	270	64.53	?	?	?	2,951	60.23	?	?	?	2,367
J	270	64.02	3.90	?	?	?	?	?	?	?	3,050
K	270	66.84	3.87	2,632	198	2,830	?	?	?	?	2,420
L	270	[illegible]	[illegible]	2,782	140	2,922	?	?	?	?	2,419
M	290	-	-	-	-	-	[illegible]	[illegible]	3,137	128	3,365
Mean		65.77	3.65	2,510	227	2,667	62.07	3.37	2,100	119	2,260
St. Dev.		3.82	0.33	621	154	630	1.32	0.25	780	42	638
COV		0.06	0.09	0.25	0.68	0.24	0.02	0.08	0.37	0.36	0.28

Notes:
#1: "?" indicates that information is not available.
#2: The gray boxes identify the lower and upper values for columns.

XX 180°

64.08 3.88 2,060 218 2,278 59.85 2.92 1,641 126 1,767

Table 3-3: Comparison of Metocean Parameters and Loads: Direction 3 (315 degree from True North)

Participant	Wave Approach Direction (from True North) (degree)	Section 17, Ultimate Strength Metocean Parameters and Loads					RP2A, 20th Edition, Metocean Parameters and Loads				
		Wave Ht., H-17 (ft.)	Current, U-17 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-17 (kips)	Wave Ht., H-20 (ft.)	Current, U-20 (ft/sec)	Wave Load on Jacket (kips)	Wave-in-Deck Load (kips)	Total Base Shear, S-20 (kips)
A	315	[illegible]	[illegible]	?	?	[illegible]	59.20	?	?	?	1,482
B	315	61.00	3.75	2,079	130	2,209	[illegible]	3.42	1,780	120	1,900
C	315	64.00	3.06	2,150	145	2,295	59.90	[illegible]	1,740	120	1,860
D	315	65.30	?	1,330	110	1,440	[illegible]	3.54	1,216	64	[illegible]
E	315	64.00	3.74	2,313	47	2,360	-	-	-	-	-
F	315	[illegible]	?	1,853	309	2,162	?	?	?	?	1,600
G	315	63.70	[illegible]	?	?	1,907	59.90	[illegible]	?	?	2,325
H	315	64.00	3.11	?	?	2,449	59.85	2.84	?	?	1,982
I	315	65.61	?	?	?	2,755	61.24	?	?	?	2,227
J	315	65.12	3.30	?	?	?	?	?	?	?	[illegible]
K	315	66.91	3.82	2,452	250	2,702	?	?	?	?	2,268
L	315	64.00	3.49	2,620	127	[illegible]	?	?	?	?	2,235
M	335	-	-	-	-	-	59.85	3.47	2,505	108	2,613
Mean		63.92	3.48	2,115	160	2,215	59.74	3.27	1,810	103	2,061
St. Dev.		3.80	0.36	429	89	488	1.43	0.35	530	27	459
COV		0.06	0.10	0.20	0.56	0.22	0.02	0.11	0.29	0.26	0.22

Notes:

#1: "?" indicates that information is not available.

#2: The gray boxes identify the lower and upper values for columns.

2,099 1,437

Table 3-4: Ultimate Strength Analysis Results: Direction 1 (225 degree from True North) -- with pile/soil considered

Participant	Base Shear: Section 17 Ult. Load S-17 (kips)	Base Shear: 20th Edition, Ref. level S-20 (kips)	Load at 1st Member with Linear IR = 1.0 S1 (optional)	Load at 1st Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/ S-20	Failure Mode/ Failure Mechanism
A (#1)	[illegible]	[illegible]	?	[illegible]	2,612	[illegible]	?
B	2,140	1,670	641	1,106	1,964	1.18	First leg/ Pile yielding
C	1,995	1,660	?	1,990	2,350	1.42	Foundation
D	1,390	1,290	1,375	?	2,623	2.03	Foundation
E	[illegible]	-	-	-	-	-	-
F	-	-	-	-	-	-	-
G	2,604	2,174	?	1,317	[illegible]	[illegible]	Pile, Soil
H	2,038	1,669	1,260	1,630	2,827	1.69	Pile/horz. braces/deck leg
I	1,992	1,600	?	1,494	2,490	1.56	?
J	?	1,948	?	[illegible]	[illegible]	1.83	Legs and braces
K	1,800	1,590	-	-	-	-	-
L	-	-	-	-	-	-	-
M	-	[illegible]	?	1,921	2,564	0.99	Soil
Mean	2,007	1,725	1,092	1,773	2,513	1.55	
St. Dev.	494	432	395	777	547	0.54	
COV	0.25	0.25	0.36	0.44	0.22	0.35	

Notes:
~~#1: Participants' RSR values differ~~
#2: The gray boxes identify the lower and upper values for columns.

2,495 2,035 - 1,560 1,747 0.86 SOIL CAPACITY

Table 3-5: Ultimate Strength Analysis Results: Direction 2 (270 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at 1st Member with Linear IR = 1.0 S1 (optional)	Load at 1st Member with NonLinear Event (optional)	Ultimate Capacity, Rn (kips)	Reserve Strength Ratio, RSR = Rn/ S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A (#1)	[illegible]	[illegible]	?	1,196	2,542	[illegible]	Pile hinge/Compression Piles
B	2,727	2,070	807	1,166	[illegible]	0.72	First leg/ Pile yielding
C	2,935	2,230	?	1,920	2,070	0.93	Foundation
D	1,590	1,310	1,500	?	2,200	1.68	Foundation
E (#1)	3,230	?	?	1,290	2,301	?	Soil capacity
F	2,234	1,713	?	1,937	1,937	1.13	Foundation/ pile double hinging
G	[illegible]	2,810	?	[illegible]	1,610	[illegible]	Pile, Soil
H	2,992	2,318	1,252	1,636	2,628	1.13	Pile, leg, horiz. braces
I	2,951	2,367	?	1,770	2,361	1.00	Jacket
J	?	3,050	?	[illegible]	[illegible]	1.03	Legs and braces
K	2,830	2,420	-	-	-	-	-
L	2,922	2,419	?	1,315	1,811	0.75	Jacket, Pile
M	-	[illegible]	?	1,900	2,446	0.75	Soil
Mean	2,667	2,260	1,186	1,582	2,219	1.08	
St. Dev.	630	638	351	416	466	0.48	
COV	0.24	0.28	0.30	0.26	0.21	0.44	

Notes:
~~#1: Participants' RSR values differ~~
#2: The gray boxes identify the lower and upper values for columns.

2,278 1,775 – 1,430 2,123 1.19

Table 3-6: Ultimate Strength Analysis Results: Direction 3 (315 degree from True North) -- with pile/soil considered

Participant	Base Shear		Load at 1st Member with Linear IR = 1.0 S1 (optional)	Load at 1st Member with NonLinear Event (optional)	Ultimate Capacity, Ru (kips)	Reserve Strength Ratio, RSR = Ru/S-20	Failure Mode/ Failure Mechanism
	Section 17 Ult. Load S-17 (kips)	20th Edition, Ref. level S-20 (kips)					
A (#1)	[illegible]	1,482	?	1,598	2,796	1.89	Pile Compression
B	2,209	1,900	847	1,197	1,861	0.98	First leg/ Pile yielding
C	2,295	1,860	?	2,100	2,430	1.31	Foundation
D	1,440	[illegible]	1,410	?	2,764	[illegible]	Foundation
E	2,360	-	-	-	-	-	-
F	2,162	1,680	?	1,905	2,545	1.51	Joint failure/ Jacket leg portal
G	1,907	2,325	?	[illegible]	[illegible]	[illegible]	Pile, Soil
H	2,449	1,982	1,274	1,554	2,895	1.46	Pile, deck legs, horz. braces
I	2,755	2,227	?	2,425	2,618	1.18	Jacket
J	?	[illegible]	?	[illegible]	[illegible]	1.19	Legs and braces
K	2,702	2,268	?	1,605	2,039	0.90	Jacket
L	[illegible]	2,235	?	1,102	1,804	0.81	Jacket, piles
M	-	2,613	?	2,174	2,613	1.00	Soil
Mean	2,215	2,061	1,177	1,831	2,446	1.25	
St. Dev.	488	459	294	691	539	0.44	
COV	0.22	0.22	0.25	0.38	0.22	0.35	

Notes:
~~#1: Participants' RSR values differ~~
#2: The gray boxes identify the lower and upper values for columns.

Component Failure Modes	A	B	C	D	E	F	G	H	I	J	K	L	M
Jacket Components													
Leg Yielding		Yes	Yes		Yes			Yes		Yes			
K-brace										Yes			
K-joint					Yes						Yes		
Horizontals					Yes			Yes	Yes		Yes	Yes	
Pile Sections:													
First Yielding	Yes	Yes						Yes				Yes	
Full Yielding		Yes											
Double Hinging	Yes					Yes	Yes						
Soil Capacity:													
Compression Capacity	YES		Yes	Yes	Yes		Yes					Yes	Yes
Tensile Capacity													

Platform Failure Mode (#1)	Pile Yielding	Pile Yielding	Pile Plunging	Foundation (Soil Capacity)	Soil Capacity	Foundation/ Pile double hinging	Pile/ Soil	Pile Yielding	Jacket	Legs & Braces	Jacket	Jacket-Pile	Soil

Note #1: Per Participants' Submittals

Figure 3-9: Comparison of Component and Platform Failure Modes - Direction 2 (270 degree from True North)

SOIL CAPACITY

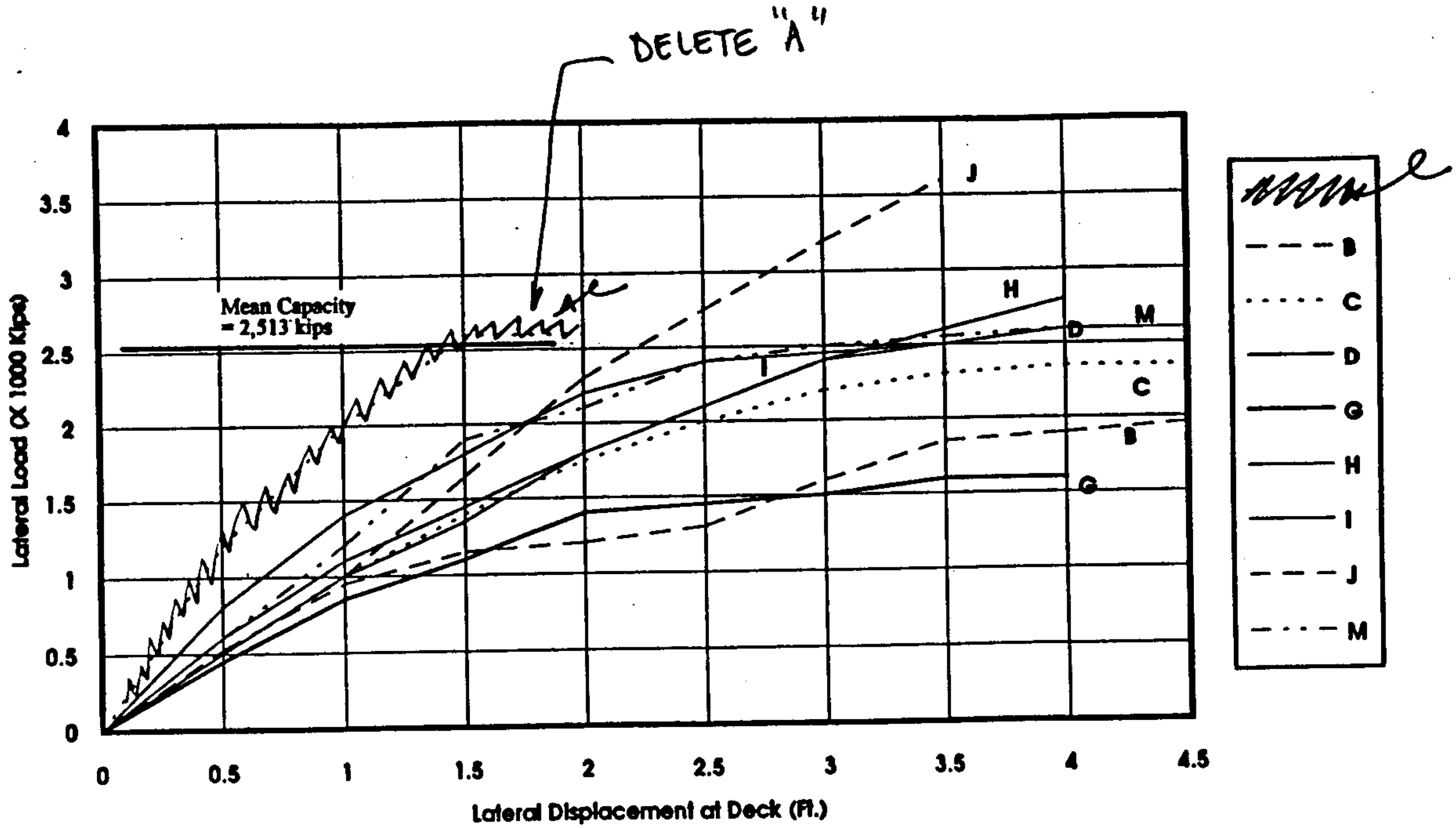


Figure 3-10 Load-Displacement Behavior - Direction 1

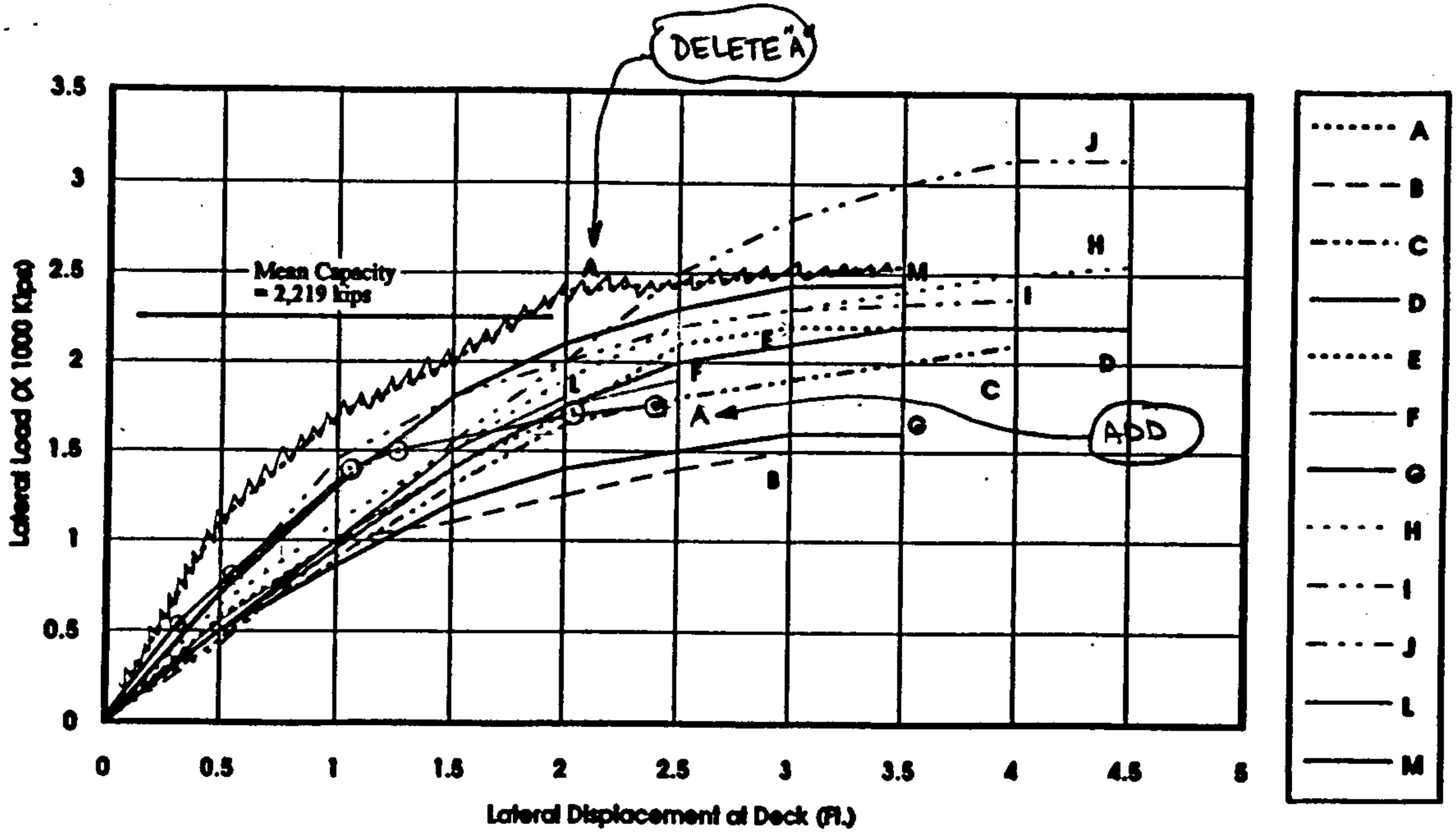


Figure 3-11 Load-Displacement Behavior - Direction 2

NOTE: SEE ATTACHED SHEET FOR DATA POINTS TO PLOT

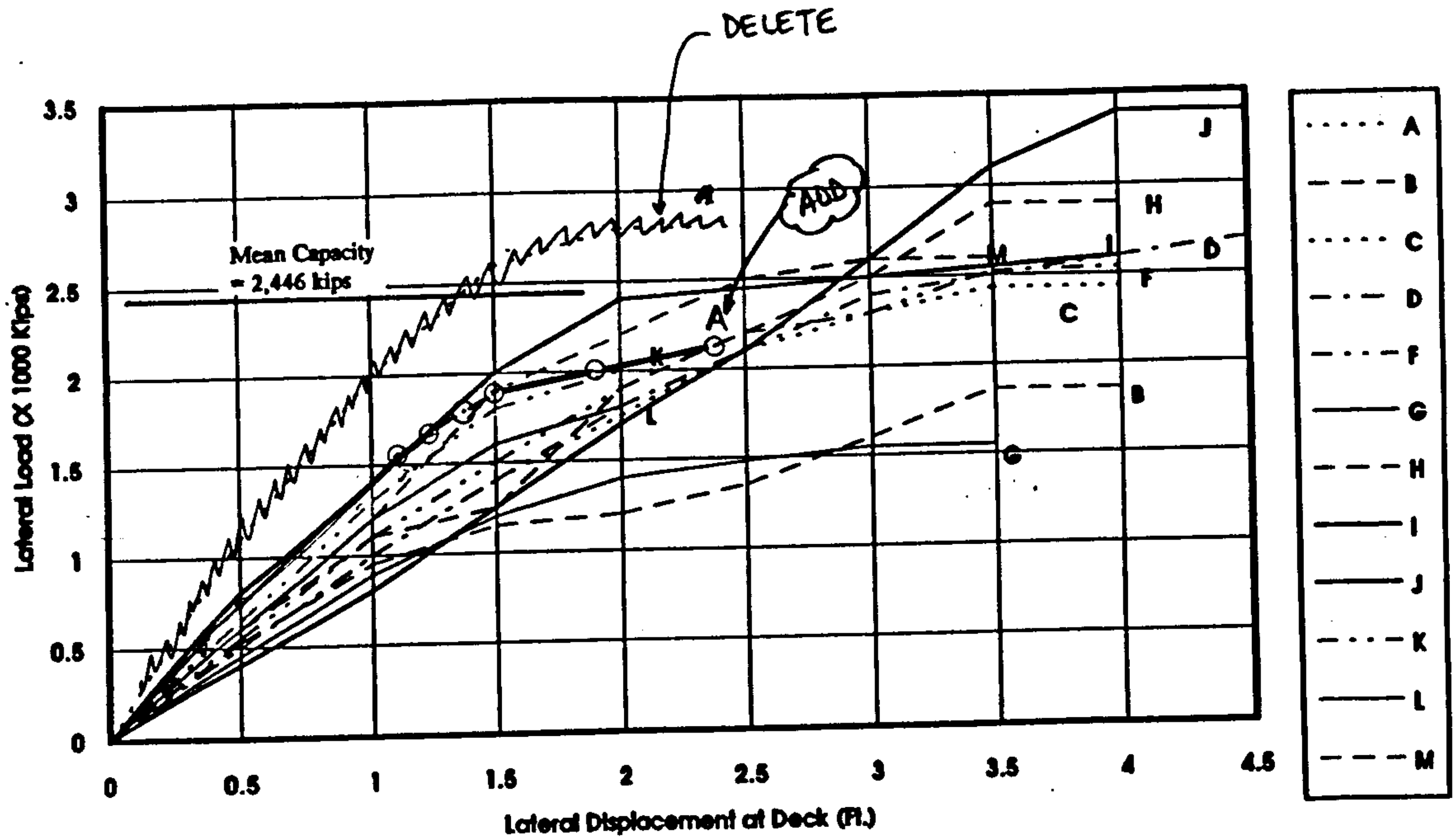
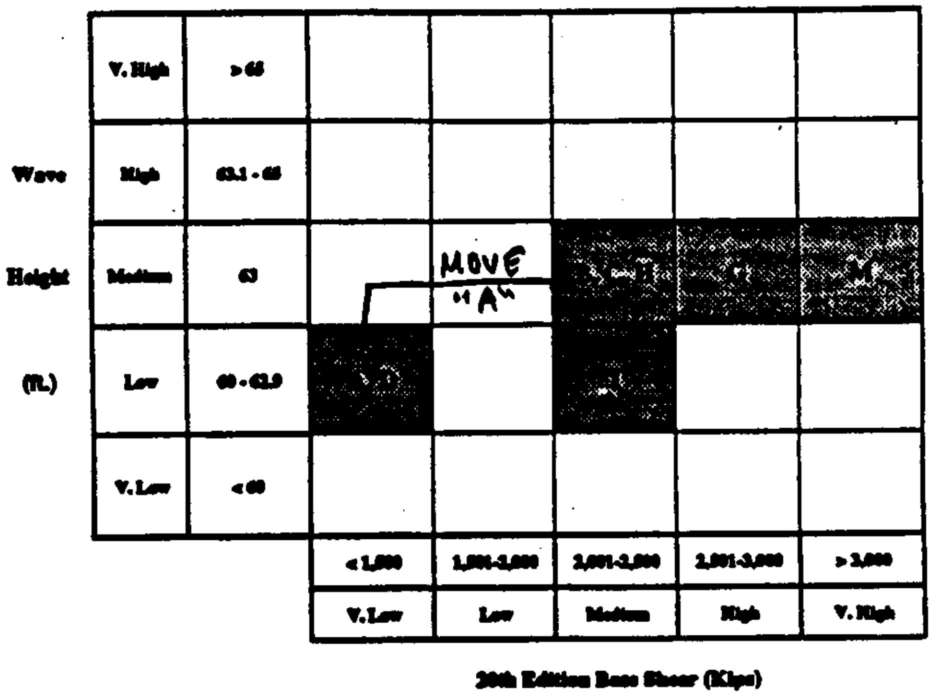


Figure 3-12 Load-Displacement Behavior - Direction 3

NOTE: SEE ATTACHED SHEET FOR DATA POINTS TO PLOT.

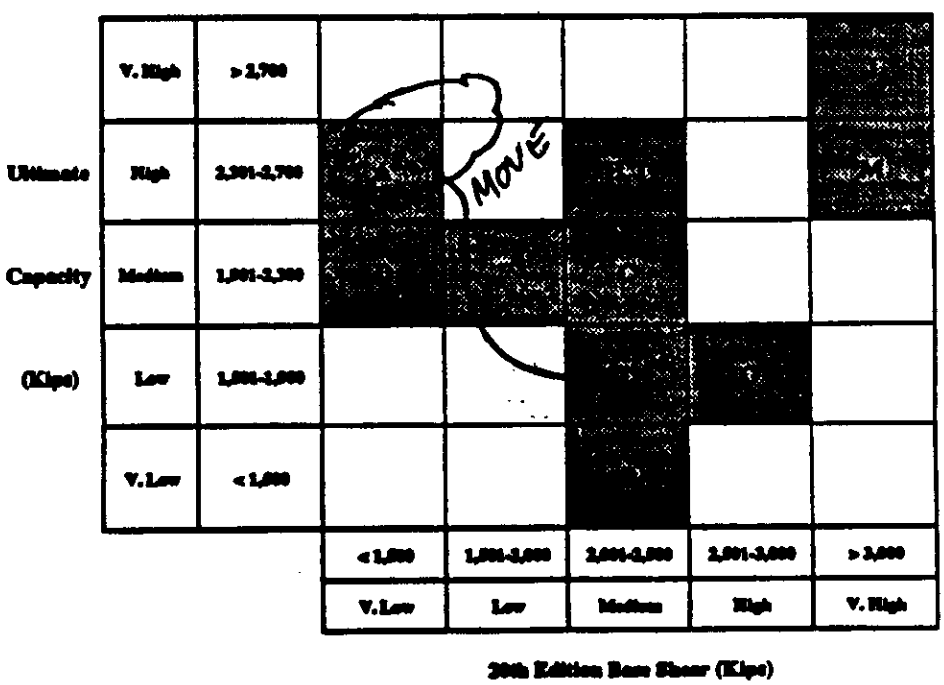
PARTICIPANT "A" DATA POINTS FOR PLOTS
(Draft Report Figures 3-11 thru 3-12)

Direction 2		Direction 3	
Lat. Load (kips)	Lat. Disp. (feet)	Lat. Load (kips)	Lat. Disp. (feet)
145	0.07	120	0.06
260	0.16	235	0.14
385	0.24	350	0.22
510	0.33	465	0.30
635	0.44	580	0.38
1,375	1.06	1,375	0.93
1,500	1.24	1,545	1.11
1,685	2.05	1,660	1.23
1,750	2.30	1,775	1.36
1,750	3.23	1,890	1.50
		2,005	1.91
		2,120	2.36



Participants E, F, J, K, L did not provide sufficient information (Ref. Table 3-2) to be included in this chart

a) Based on Selected Wave Height (20th Edition) and Base Shear



Participants E and K did not provide sufficient information (Ref. Table 3-6) to be included in this chart

b) Based on Reference Level Base Shear and Ultimate Capacity

Figure 3-20: Classification Based on Wave Height and Analysis Results-Direction 2

PARTICIPANT "B" RESUBMITTAL

PART A: Benchmark Analysis

A.1: Environmental Criteria

Analyses were performed in three wave directions aligned to platform broadside, endon and diagonal directions. The environmental criteria used to generate pushover load patterns in the these wave directions are given in Table A.1.1. API Section 17 environmental criteria for ultimate strength analyses are given in table A.1.2 for comparison. Table A.1.3 illustrates the 20th edition 100 year storm criteria. The pushover load pattern generated was incrementally applied on to the structure either until collapse due to significant nonlinear events or program termination due to analysis parameters reaching specified limits.

Table A.1.1: Environmental Criteria Used for Analyses

Wav Dir	W.R.T Tr. Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	225	75	13.5	3.747/C	3	143	API Sect. 17	827	3692
End-on	315	75	13.5	1.941/C	3	143	API Sect. 17	653	2980
Diag	270	75	13.5	3.359/C	3	143	API Sect. 17	557	3168

Table A.1.2: Section 17 Ultimate Strength Environmental Criteria

Wav Dir	W.R.T Tr. Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	225	61	13.5	3.747/C	3	143	API Sect. 17	130	2209
End-on	315	64.6	13.5	1.941/C	3	143	API Sect. 17	319	2140
Diag	270	68	13.5	3.359/C	3	143	API Sect. 17	486	2727

Table A.1.3: API 20th Edition 100 yr Environmental Criteria

Wav Dir	W.R.T Tr. Nth (Deg)	Wav. Ht. (Feet)	Wav. Period (Sec.)	Curr Spd &Profile (Ft/Sec)	Storm Surge (Feet)	W.Spd @10 m (Ft/Sec)	Dk. Frc Appr.	Deck Force (Kips)	Toatal B.S. (Kips)
Broad Side	225	56.7	13	3.424/C	3.5	135	API Sect. 17	120	1900
End-on	315	59.85	13	1.772/C	3.5	135	API Sect. 17	125	1670
Diag	270	63	13	3.069/C	3.5	135	API Sect. 17	118	2070

In addition to the criteria given in Table A.1.1, a wave kinematics factor of 0.88 was used in all wave directions. Current blockage factors of 0.8, 0.8, 0.85 were also used respectively in broadside, endon, and diagonal directions. Marine growth was assumed to be 1.5 feet constant to 150 feet below water line. Drag and inertia coefficients were selected according to API 20th edition recommendation. Wind forces were computed using the projected wind areas and the wind speeds given in Table A.1.1 as recommended by API 20th edition. Wave encounter and current stretching were not considered in the final analyses since they were found to have insignificant effect on the applied base shear in preliminary analyses.

The wave directions chosen in analyses correspond to the largest environmental forces on the platform with respect to platform primary axes. Due to this reason and due to platform symmetry, ultimate capacity of the platform determined in these directions will adequately represent the overall critical strength of the platform. Therefore, it was unnecessary to perform analyses in eight primary directions as suggested by API RP 2A Section 17.

A.2: 3-D Model Generation

The 3-D model of the structure was generated using the 1-D elements available in PMB's Capacity Analysis Program (CAP). The type of elements assigned to each component of the jacket are as follows:

Vertical Braces:	Marshall Struts	Inelastic Buckling
Horizontal Braces:	Beam-Column/	Lumped Plasticity Material Model
	Marshall Struts	Inelastic Buckling
Legs/Piles:	Beam-Column	Lumped Plasticity Material Model
Cond. Framing:	Linear Beam-Column	
Boat Landings:	Wave Load	No Stiffness
Deck Area:	Wave Load	No Stiffness
Deck Members:	Linear Beam	No Wave Loading
Soil:	PSAS	Inelastic as per API

The material yield strength was assumed to be 42 ksi. Effective length factor (K) for all vertical braces was assumed to be 0.65. For all other members, K factor was chosen according to API RP 2A 20th edition recommendations. The capacity of the jacket components were estimated based on the API RP2A-LRFD equations with the load and resistance factors set to 1.0. Equations in Section D.2.2.1, and Section D.2.3 respectively, were used to estimate the buckling and bending capacity of the members. The joint capacity was estimated based on procedures and equation outlined in Section

E.3.1.1 by assuming that the parameter Q_f equals 1.0. Nevertheless, it should be noted that the joint capacities based on API equations have been shown to be conservative. However, without sufficient experimental results to backup the physical observations presently available, it is difficult to consistently predict joint capacities contrary to that result from API equations.

The joint cans on the leg members were not explicitly modeled. The thickness for the entire length of the members was assumed to be the same as that at the midsection. The beam-column elements used to model the leg members assumes lumped plasticity model that allows plastic hinges to form at the end nodes only. Therefore, in the event of failure modes initiated by legs yielding, using these elements to model the legs without considering joint can thickness can cause premature yielding of those members. Premature yielding of leg members could result in erroneous nonlinear behavior of the structure. However, neglecting joint can thickness will not significantly affect the structural performance if its failure is controlled by modes other than the leg members yielding. To correctly model joint can thickness in CAP, one additional element at each end of leg members need to be introduced. This technique may substantially increase run time and possibly cause numerical difficulties. Since member failure modes of most K-braced structures are controlled by the braces buckling, neglecting leg joint cans in the member strength modeling is not expected to significantly affect the overall structural behavior. However, if the results indicate failure patterns on the contrary, this modeling assumption will need to be re-visited.

Modeling joint behavior has been a difficult task. Results from past analyses have shown that some of the techniques used gave questionable results (Andrew JIP, Phase 1). It has been proposed that joint modeling techniques should be studied carefully with some experimental backup. For these reasons, the joint behavior was not considered in the modeling. Nevertheless, two modeling techniques were considered in preliminary analyses. These techniques and the findings of the analyses are briefly discussed in Section A.5.

Only the primary members on the deck were explicitly modeled. Dead loads due to secondary members and equipment on each deck levels were applied as appropriate concentrated loads. All lateral forces on the deck due to wave were modeled by wave load elements representing equivalent areas computed based on the "silhouette approach" recommended in Section 17 of API RP 2A (provided by PMB). The drag and inertia coefficients for the deck area members were selected according to specifications provided by PMB. Boat landings were modeled as representative equivalent area wave load elements. Wave load elements do not provide any stiffness to the structure. Wind loads calculated based on API recommendations were applied as appropriate concentrated loads on deck-legs.

Piles were ungrouted. Legs and piles were connected by shim elements at each member end node. Shim elements have zero stiffness in the pile direction and provide high rigidity in the plane normal to the pile direction. This configuration forces the pile and casing nodes to move together in the lateral plane but slip freely along the pile direction. Conductors were modeled as linear beam elements with vertical and rotational restraints at the mudline. At intermediate conductor framing, deflections of conductor nodes were slaved in the global X and Y direction to appropriate nodes on the framing. Slaving deflections will provide a rigid link in the slaved directions between the master and the slaved nodes.

Soil was modeled by PSAS elements. PSAS elements represent the soil resistance by two orthogonal springs laterally and one spring axially. The lateral and the axial spring properties were based on the P-Y, T-Z and Q-Z curves generated based on API RP 2A 19th edition guidelines and static soil properties. The 20th edition changes affect only the T-Z curves generated in the inelastic regime. The automated feature that CAP has for this purpose is not capable of handling the 20th edition changes to the T-Z curve at the moment. Since the effects of the changes to the T-Z curves are insignificant on the overall ultimate capacity of the platform, using the 19th edition T-Z properties will suffice for the present purpose. It should be noted that for the diagonal direction wave considered, the lateral springs were rotated so that one spring is in the direction of the wave. Rotating the springs in the direction of the wave more realistically represent the soil resistance in the principle direction of lateral deformation.

A.3: Software Description

As mentioned earlier, the analyses were performed using PMB's CAP program. CAP essentially is a graphic interface to the finite element program SEASTAR. The graphical preprocessor provides users with an effective tool to build or modify a 3D structural model. The post processor allows users to check analysis results effectively and to present results with several graphical options.

The finite element library in SEASTAR program contains several inelastic and elastic type of elements. In a typical static pushover analysis, inelastic beam-column, Marshall strut, nonlinear truss and linear beam elements are used to model the jacket while PSAS and beam-column elements represent the nonlinear foundation. The deck members are usually modeled by linear beam elements. Wave induced deck forces and boat landings are modeled by wave load elements. The beam column elements in SEASTAR do not have inelastic buckling capability. The Marshall strut is capable of modeling inelastic buckling, however, it does not carry moments. All the inelastic elements in SEASTAR are capable of modeling fracture behavior.

The wave load is typically applied as a load pattern which is generated based on given wave and current profile. Typical load pattern generation is based on regular waves. The load pattern is applied incrementally as nodal loads until platform failure is reached.

CAP provides Preconfigured macros to generate pushover load patterns and to perform static pushover analyses, among others. In most cases, the default parameters set in these macros are adequate to perform the necessary analyses. Fine tuning these parameters may result in more efficient analysis runs. However, changing the analysis control parameters in the static pushover macros is recommended only for those with significant experience in performing nonlinear analyses.

The new static pushover solution algorithm available in SEASTAR is quite efficient compared to the pseudo static algorithm from the older versions. Nonetheless, with the default settings for the analysis parameters, the analysis runs averaged six to seven hours for each direction considered on a SUN SPARC10 machine. However, it should be noted that some of the analyses probably could have been terminated much before the default 300 load steps which would have resulted in significant reduction is run time.

A.4: Ultimate Strength Analysis Results

This section addresses the ultimate strength results that is required to be presented in the JIP report. Results from analyses performed by using a full nonlinear model are presented in Tables A.4.1, A.4.2 and A.4.3, respectively, for broadside, endon and diagonal wave directions. Lateral load/displacement curves are presented in Figures A.4.1 through A.4.3 for the respective wave directions. The lateral displacements plotted refer to the node (1219) at deck level +43 feet at the SE corner leg intersection. The lateral loads refer to the base shear induced by the pushover load generated based on the environmental conditions given in Section A.1. The base shear was computed in a horizontal plane at elevation -158 feet (1 feet below mudline). Figures A.4.4 through A.4.6 illustrate the component failure sequence and the deformed shape (magnified) of the structure at analysis termination.

A.5: Commentary on Ultimate Strength Analysis Results

The analyses results presented here indicate that in all three wave directions, the failure of the platform is primarily governed by the piles yielding and eventually plasticizing. In all three directions, the first components to reach yield were the leg members in the bottom bay. The piles connecting to the pile head nodes and the leg-pile members in the bottom leg members yielded following the leg members. The pile section at 90' below mudline where the wall thickness reduces from 1.875 inches to 1.5 inches fully plasticized for deck deformations in the order of 3 feet. All members that yielded were dominated by bending stresses, as expected. No other components in the jacket reached their yield or buckling capacities. The largest axial forces were in the bottom bay vertical braces which were over 100 kips less than their buckling capacity.

The type of failure observed in the analyses indicate extremely weak foundation strength. This behavior is not entirely unexpected since the top 100 feet of soil layer is soft clay with a shear strength profile varying from 80 pcf at 0 feet to 600 pcf at 100 feet. Failure was most likely initiated by the large lateral deformation of the pile sections near mudline due to insufficient lateral resistance provided by the weak top layer of soil. As listed in Section A.2, the legs and piles were modeled by using beam-column elements which have strain hardening properties in the nonlinear regime. Since no braces buckled in the structure, the nonlinear behavior of the structure was entirely governed by the yielded legs and piles. Due to the strain hardening behavior of the yielded legs, there was no reduction in lateral load capacity of the structure in the nonlinear regime. In this event, the analytical lateral load capacity of the structure increases until the analyses terminate due to numerical difficulty or complete collapse of the structure due to excessive deformation. Under these circumstances, a reasonable estimate of the ultimate capacity was made as the lateral load at which the first pile section fully plasticized.

As discussed in Section A.2, joint cans on the legs were not explicitly modeled. This modeling assumption is based on behavior of typical K-braced jackets where the ultimate capacity is governed by braces buckling. However, the results observed here was quite different from that originally anticipated. Therefore, an analysis with the leg thickness increased to account for the joint cans was performed in the broadside wave directions to verify the significance of this modeling difference. The results showed no significant change in the ultimate capacity in broadside direction. The mode of failure remained controlled by piles or legs yielding except that the first members to yield were the top pile sections instead of the bottom bay leg members as in the earlier analysis. It was inferred that the results in the other two wave directions would also show similar tendencies. Thus, the modeling assumption to neglect joint cans in the analyses is acceptable.

In an attempt to model joint behavior, joints that were weaker than the braces (strength less than buckling capacity) were intended to be modeled by using equivalent nonlinear truss members. These truss members have yield capacity equivalent to joint strength. Joints are modeled by replacing the braces with these equivalent truss elements. The post yield behavior of the joints may be modeled either as strain hardening or fracturing types. It should be noted, however, that the fracture modeling of joints is a complex issue and there is insufficient experimental data to back up any assumptions made in using this technique. One example of the complexity of this issue is the determination of the strain at which fracture initiates. The selection of the fracture strain essentially controls the prediction of the joint strength and thus will significantly affect the ultimate capacity of the jacket. Since proper experimental data are unavailable, the fracture strain will be assumed to be equal to the yield strain for any fracture type joint modeling considered here. Therefore, at reaching yield capacity, the equivalent joint members would be completely severed from the chord member. Thus, no brace support will be provided to the bays at which joint failures occur. This failure mode typically leads to a lower bound value of the ultimate capacity. Strain hardening approach on the other hand, may not allow correct redistribution of the forces in the structure after the joints yield. Since load shedding due to failure of joint is not captured by this type of modeling, the resulting ultimate capacity can be over estimated. Local buckling effects and joint flexibility were not considered in modeling.

Upon investigating K-joint capacities, it was found that they were stronger than the braces that frame into them. Under this circumstance the braces will have buckled before joints reached capacity. Therefore, it was not necessary to use either one of the techniques discussed above to model "weak joints" in any of the analyses.

In another analysis performed to verify the load path dependency of the structure, API RP 2A Section 17 ultimate strength environmental criteria were used in the broadside direction. The wave height of 61 feet in the latter analysis produced only nominal deck loads unlike the runs corresponding to the 75 feet wave in the earlier analysis. The pushover load generated was incrementally applied as in other analyses. The failure path and the ultimate capacity from this analysis had no significant changes from the earlier analysis.

Finally, analyses were performed to verify the effect of fixing the base of the platform. Results from these analyses in broadside and diagonal directions are presented respectively, in Figure A.5.1 and Figure A.5.2.

PART B: Review and Feedback to the API TG 92-5

B.1: Ultimate Strength Analysis Criteria- Environmental

In Figure 17.6.2-4, the caption should indicate that the directions and factors also apply to currents.

B.2: Structural Assessment

Guidelines to select suitable analysis method (linear global, local overload or global inelastic) given in Section 17.7.3a through 17.7.3c should be more clearly stated.

Item 3.b and 3.c in Section17.7.3c do not address the issue of modeling braces that carry significant moments. One example is braces that frame into pile heads.

Item 3.d in Section 17.7.3c does not clearly state what the actual loads or the loads based on the strength that act on joints. Some joint modeling techniques should be stated here with their advantages and disadvantages.

B.3: Commentary of Ultimate Capacity Evaluation

See Section A.5

Table A.4.1: Ultimate Strength Analysis Results for Broadside Wave (225° TN)

Lateral load level for first member with unity check =1.0# **847** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
76	2.005	1197	Leg Members A1-1, A2-18, B1-35, B2-52	Initial Yield	Bottom bay leg Members - First component to fail B/C element first yield @ node I
78	2.068	1221	Pile Members B2-455, B2-456	Initial Yield	SE and SW piles at 90' below mudline (ML) B/C elements first yield @ node J
80	2.252	1287	Pile Members B2-451,452,457,458,501, 502,509,510	Initial Yield	Pile sections at 90-100' below ML and at ML on all 4 piles B/C elements first yield @ node J
90	2.413	1334	Pile Members B2-451,455,456,462	Initial Yield	Pile section at 90-100' below ML B/C elements first yield @ nodes I&J
96	2.542	1355	Leg-Pile Members LpileA1-2, LpileA2-19	Initial Yield	Bottom bay leg pile members B/C element first yield @ node I
210	3.269	1776	Level 1 horizontals Lev1H-126,154	Initial Yield	Level 1 horizontal B/C element first yield @ node I
286	3.760	1861	Pile Member PileB2-453	Fully Plastic	NE corner pile at 90' below ML. Lateral load taken as ultimate capacity

Table A.4.2: Ultimate Strength Analysis Results for Endon Wave (315° TN)

Lateral load level for first member with unity check =1.0#   **641** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
77	1.974	1186	Leg Members LegB1-35, LegB2-52	Initial Yield	Bottom bay leg members - First component to fail B/C elements initial yield @ node I
78	2.014	1203	Leg Member LegA1-1	Initial Yield	Bottom bay leg members B/C elements initial yield @ node I
79	2.045	1215	Leg, Pile members LegA2-18, PileB2-458	Initial Yield	Bottom bay leg member and pile section on NW cornr pile at 90' below ML B/C elements initial yield @ node I
82	2.277	1299	Pile Members PileB2-457,462,503,512	Initial Yield	Pile sectins at 90-100' below ML and at ML B/C elements initial yield @ node J
92	2.509	1352	Leg Pile Member LpileB2-36	Initial Yield	Leg pile B2 in bottom bay B/C elements initial yield @ node I
114	2.519	1330	Pile Members PileB2-455,456	Initial Yield	Pile members at 100' below ML B/C elements initial yield @ nodes I&J
204	3.019	1603	Horizontal Member Lev1H-71	Initial Yield	Horizontal member at level 1 B/C elements initial yield @ node I
453	4.360	1949	Pile Member PileB2-454	Initial Yield	NW pile section 100' below ML B/C element initial yield @ nodes I&J
486	4.402	1964	Pile Member PileB2-454	Fully Plastic	NW pile section 100' below ML Lateral load taken to be ultimate capacity

Table A.4.3: Ultimate Strength Analysis Results for Diagonal Wave (270° TN)

Lateral load level for first member with unity check =1.0# **807** Kips

Load Step	Lateral Disp at Deck Level (+43') at S.E. Leg Ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
91	1.898	1166	Leg Member LegA1-1	Initial Yield	Bottom leg section on leg A1 - First component to fail B/C element first yield @ node I
92	1.996	1219	Leg and Pile Members LegB1-35, PileB2-501,509	Initial Yield	Bottom bay leg and piles at ML B/C elements first yield @ node I
93	2.055	1248	Leg Members LegA2-18, LegB2-52	Initial Yield	Bottom leg sections on legs A2, B2 B/C elements first yield @ node I
94	2.106	1271	Leg Pile Member LpileA1-2	Initial Yield	Bottom pile member in leg A1 B/C elements first yield @ node I
98	2.361	1376	Pile Members PileB2-456,503,512	Initial Yield	Pile section at ML and at 100' below ML B/C elements first yield @ node I or J
128	2.533	1397	Pile Member PileB2-456	Initial Yield	SE pile at 100' below ML B/C element first yield @ nodes I & J
131	2.545	1398	Pile Member PileB2-451	Initial Yield	SW pile at 100' below ML B/C element first yield @ nodes I & J
224	2.938	1496	Pile Member PileB2-456	Fully Plastic	SE pile at 100' below ML Lateral load taken as the ultimate capacity

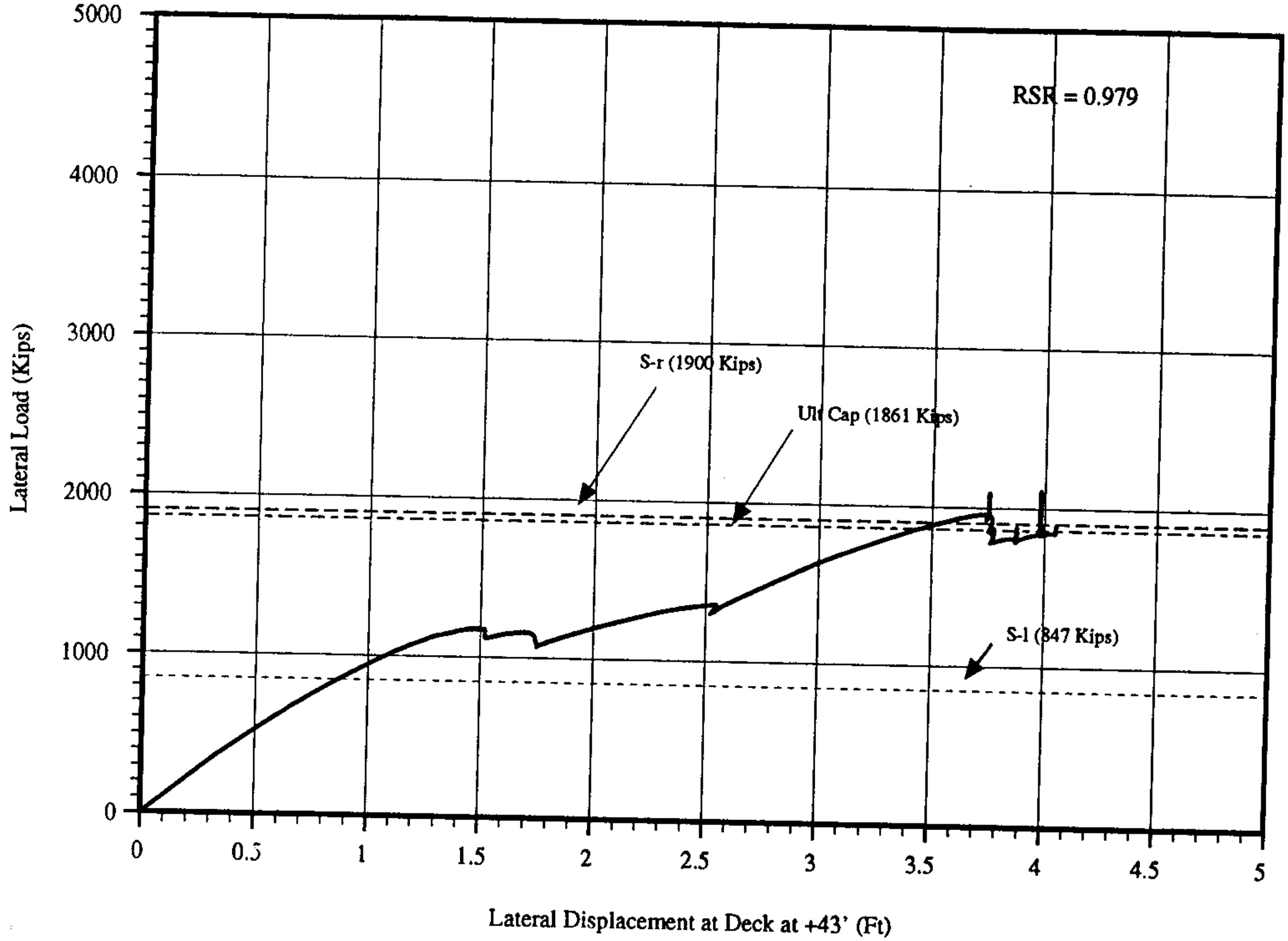


FIGURE A.4.1: Pushover Analysis Results - Broadside Wave (225° TN)

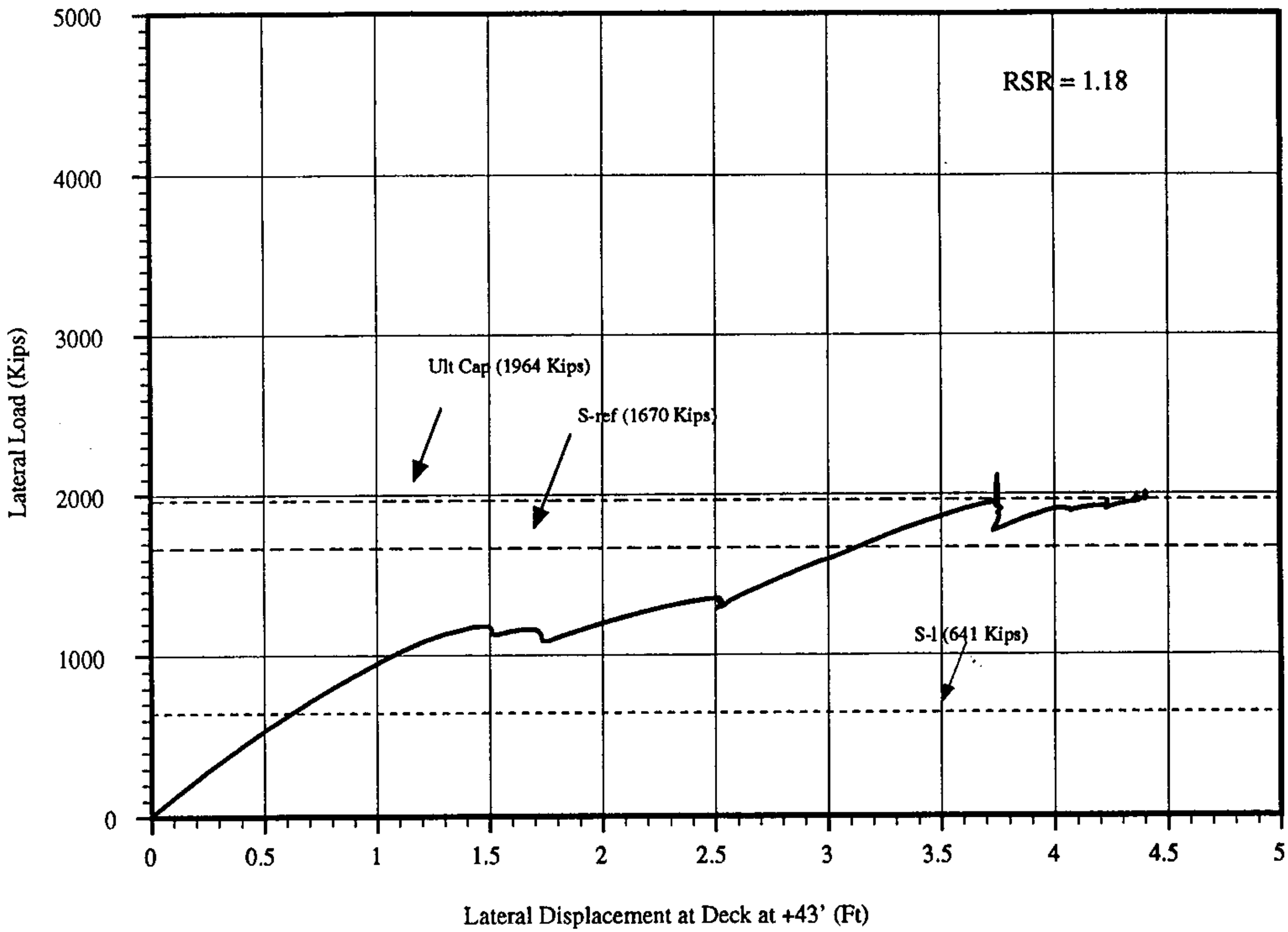


FIGURE A.4.2: Pushover Analysis Results - Endon Wave (315° TN)

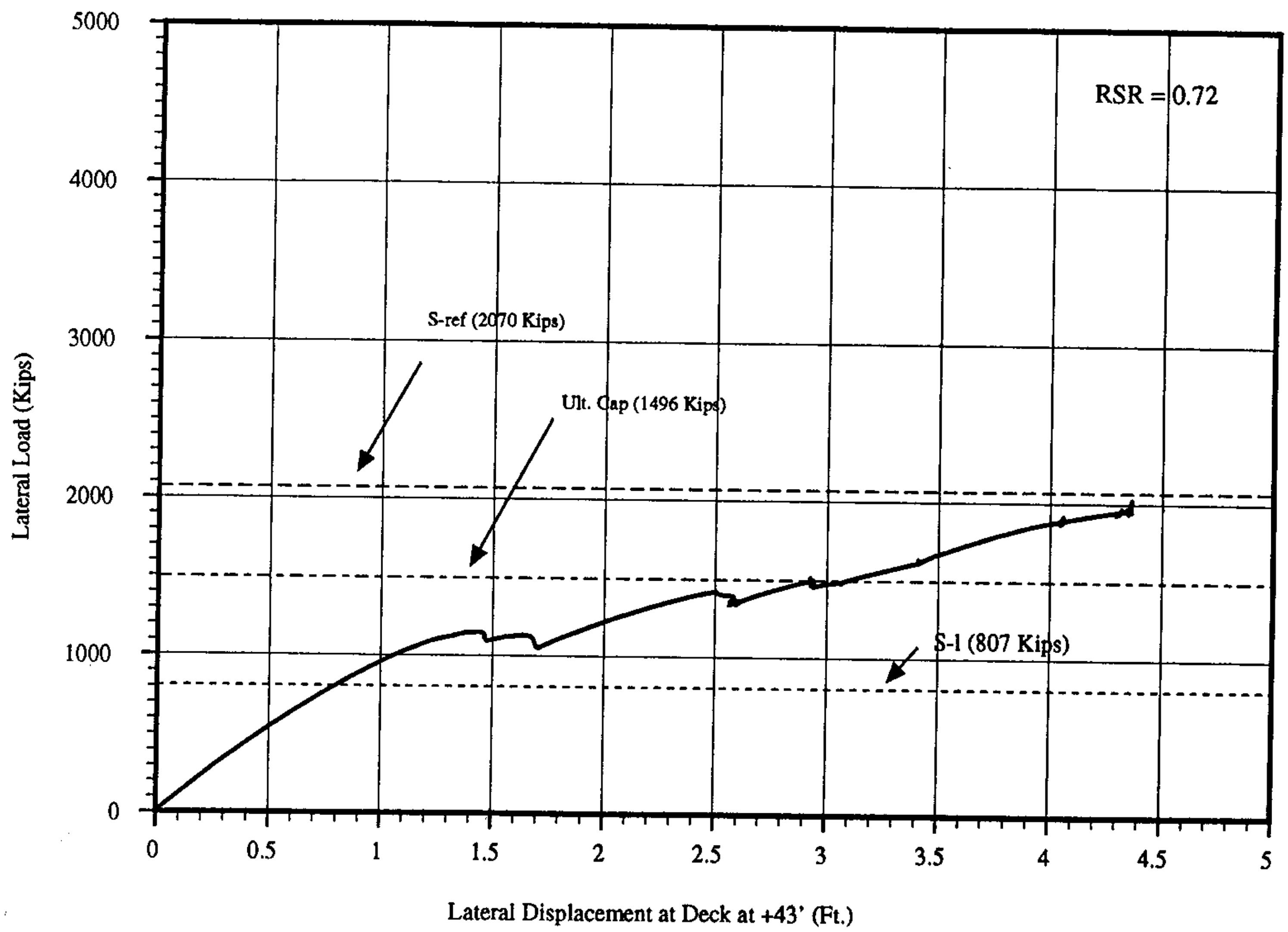
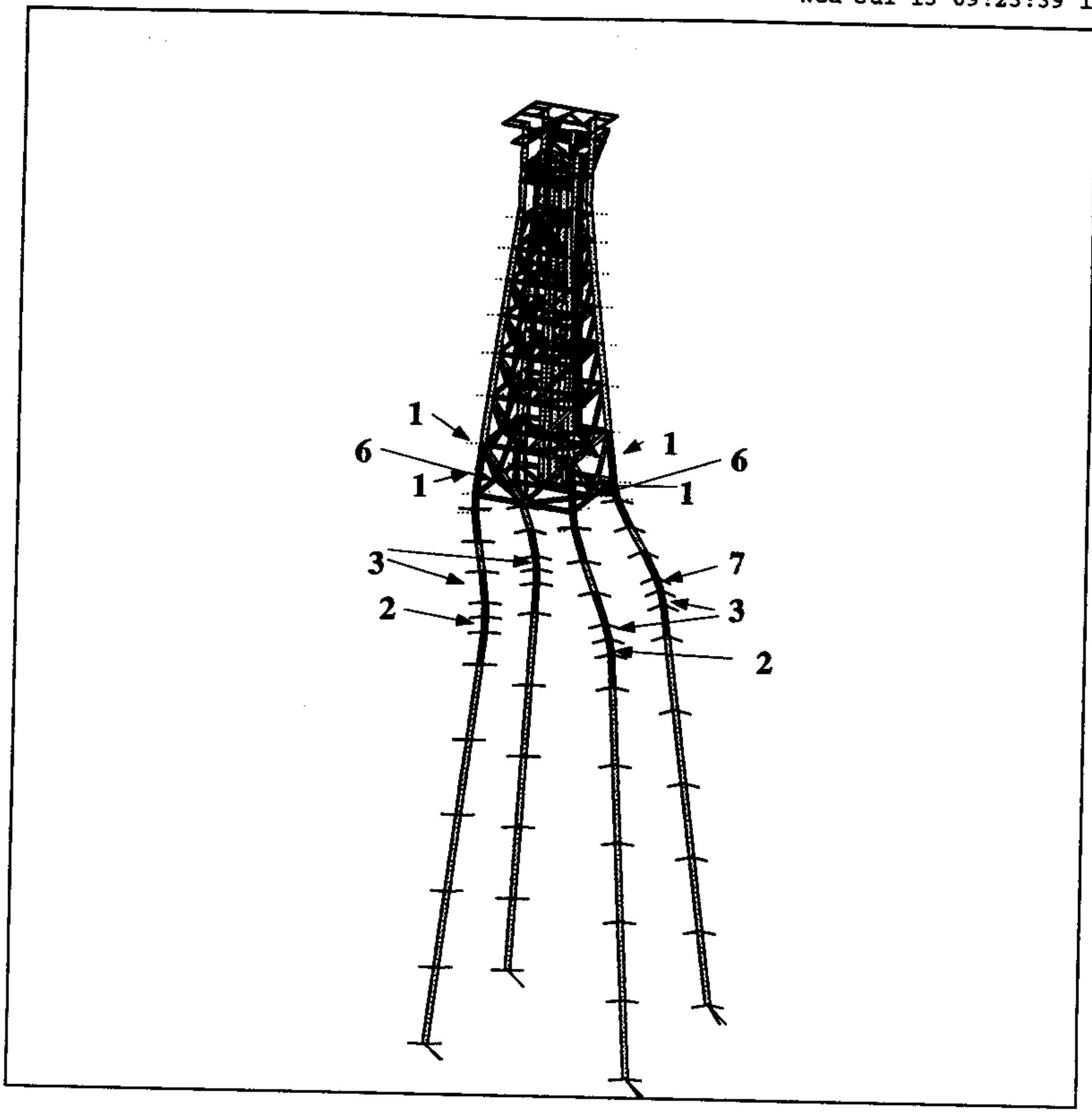


FIGURE A.4.3: Pushover Analysis Results - Diagonal Wave (270° TN)

Project: bmark Model: pushy Version: 42 Wed Jul 13 09:23:39 1994 *



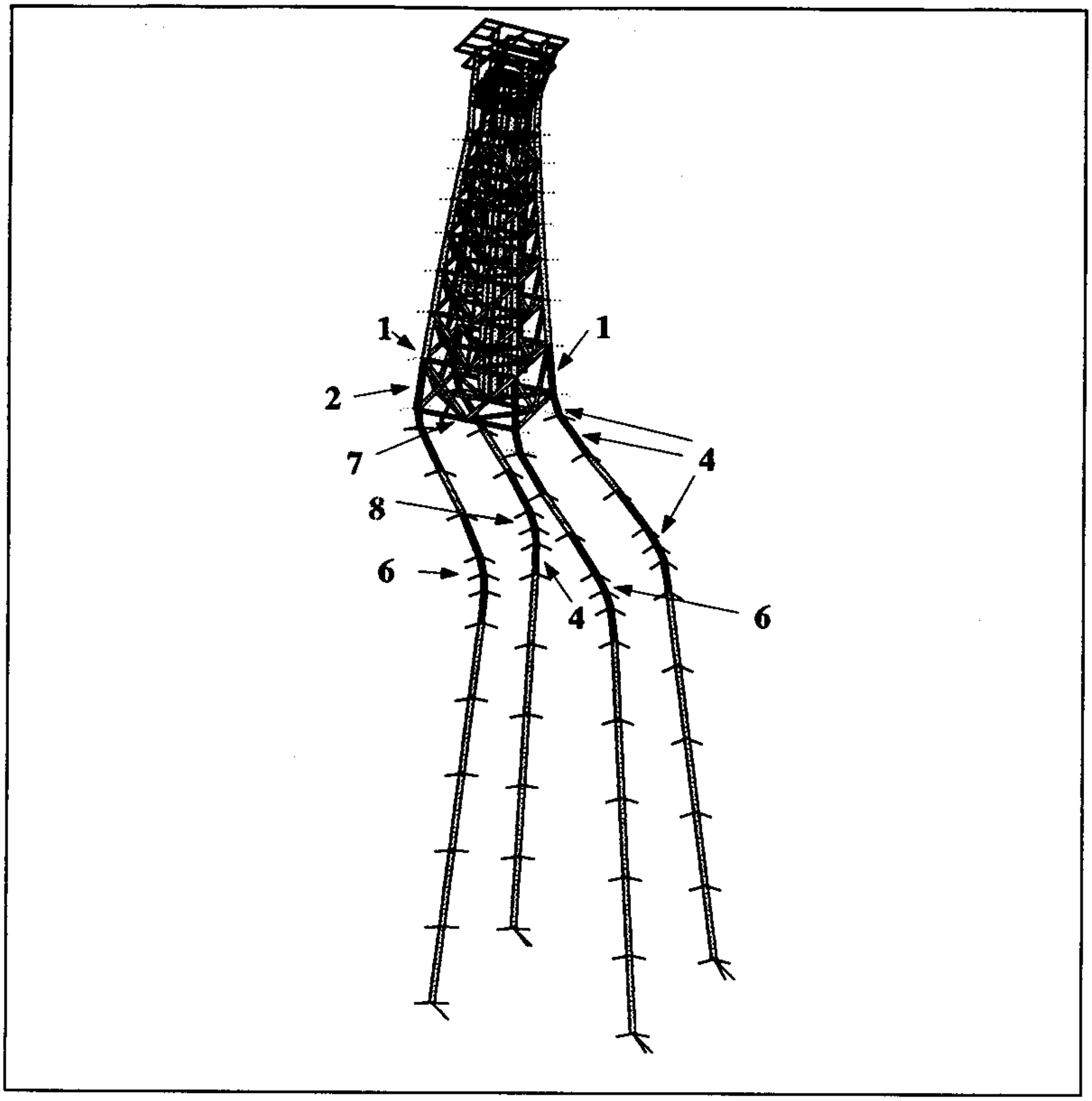
CAP Final Deflected Shape (Broad Side - Mag 10)

Inelastic Events Legend

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

FIGURE A.4.4: Deflected Shape at Analysis Termination and Failure Sequence - Broad-side Wave (225° TN)

Project: bmark Model: pushx Version: 12   Mon Jul 11 13:49:23 1994 *



CAP Final Deflected Shape (Magnification 10)

Inelastic Events Legend

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

FIGURE A.4.5: Deflected Shape at Analysis Termination and Failure Sequence - End-on Wave (315° TN)

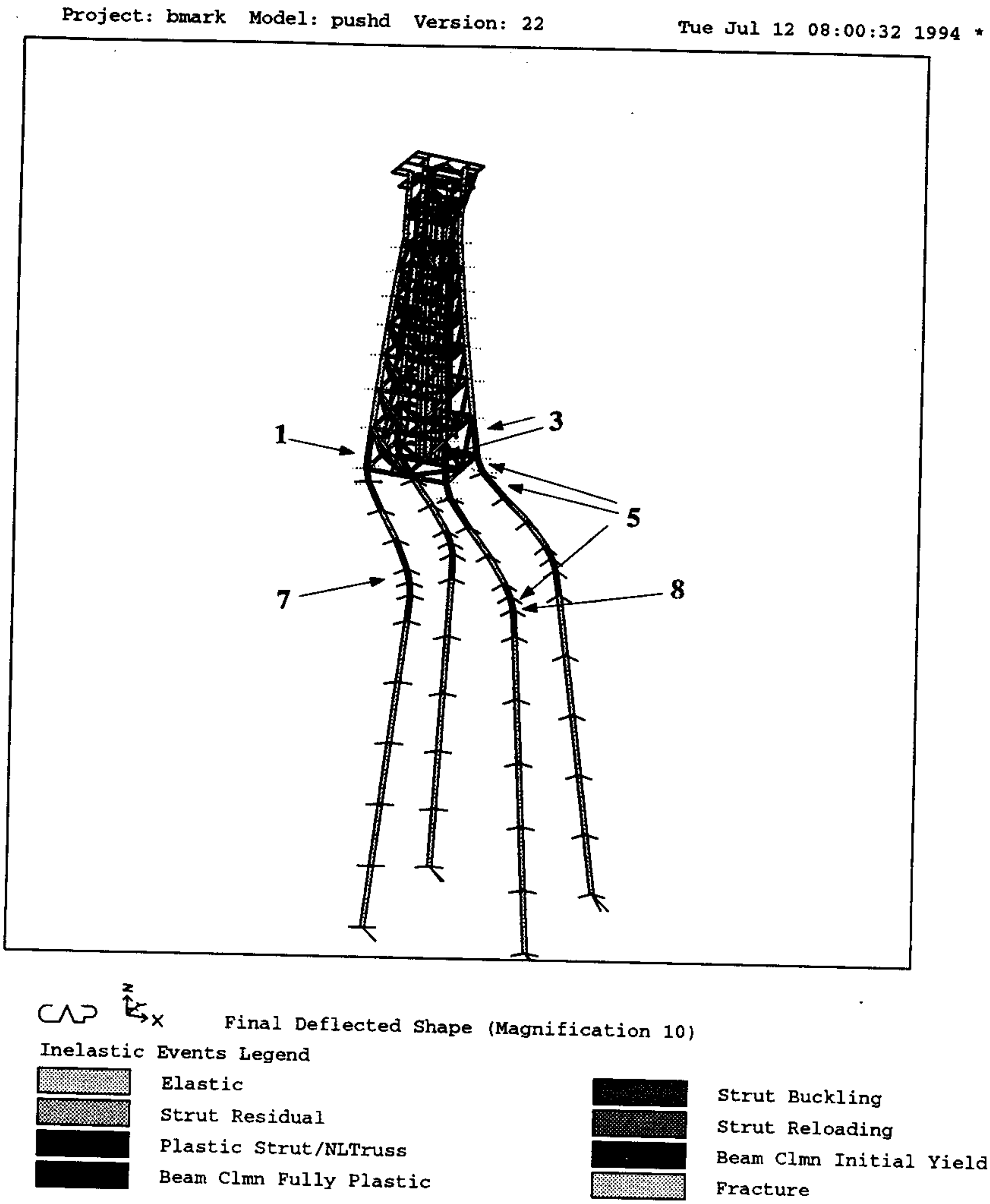


FIGURE A.4.6: Deflected Shape at Analysis Termination and Failure Sequence - Diagonal Wave (270° TN)

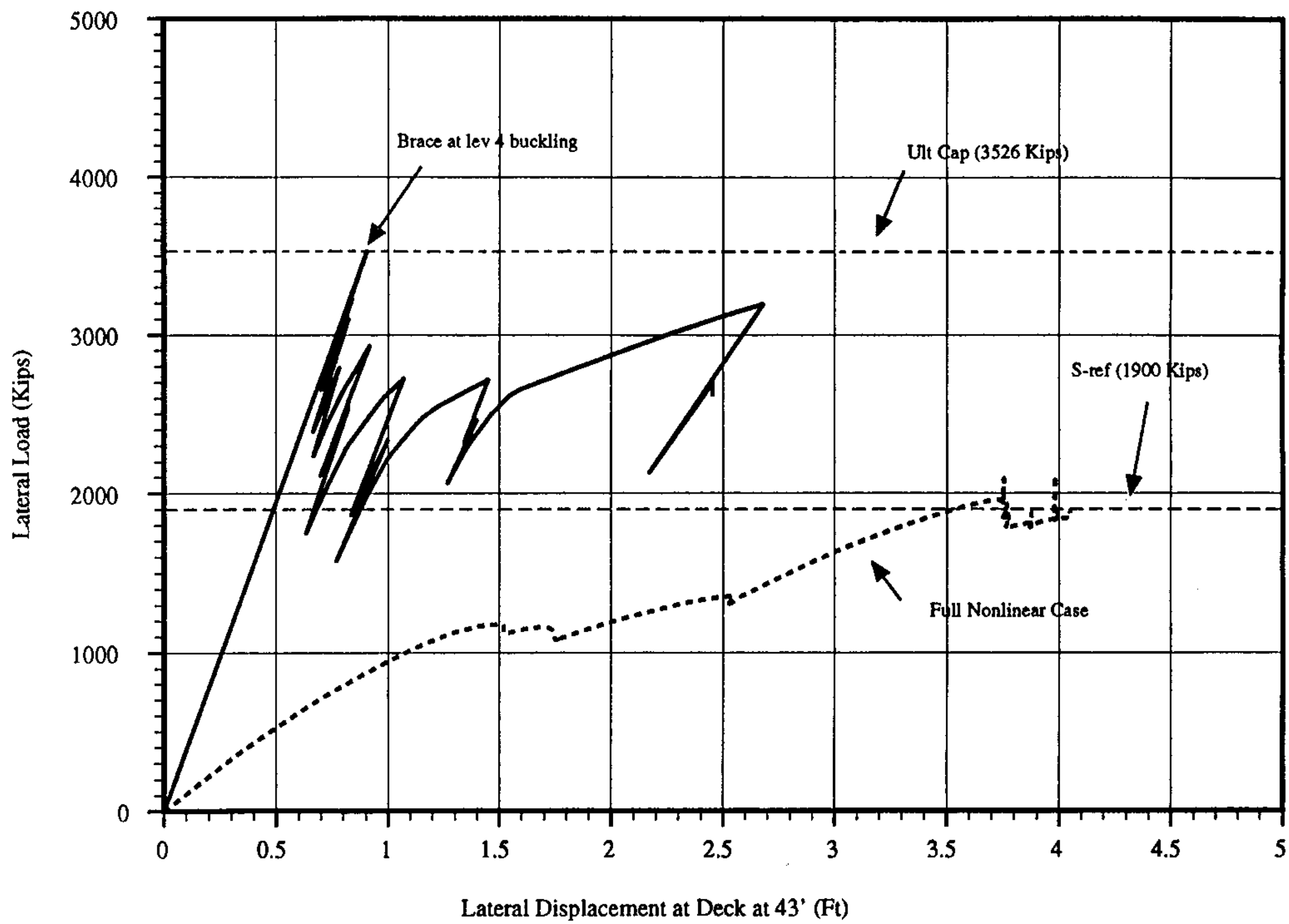


FIGURE A.5.1: Fixed Base Case Results - Broadside Wave (225° TN)

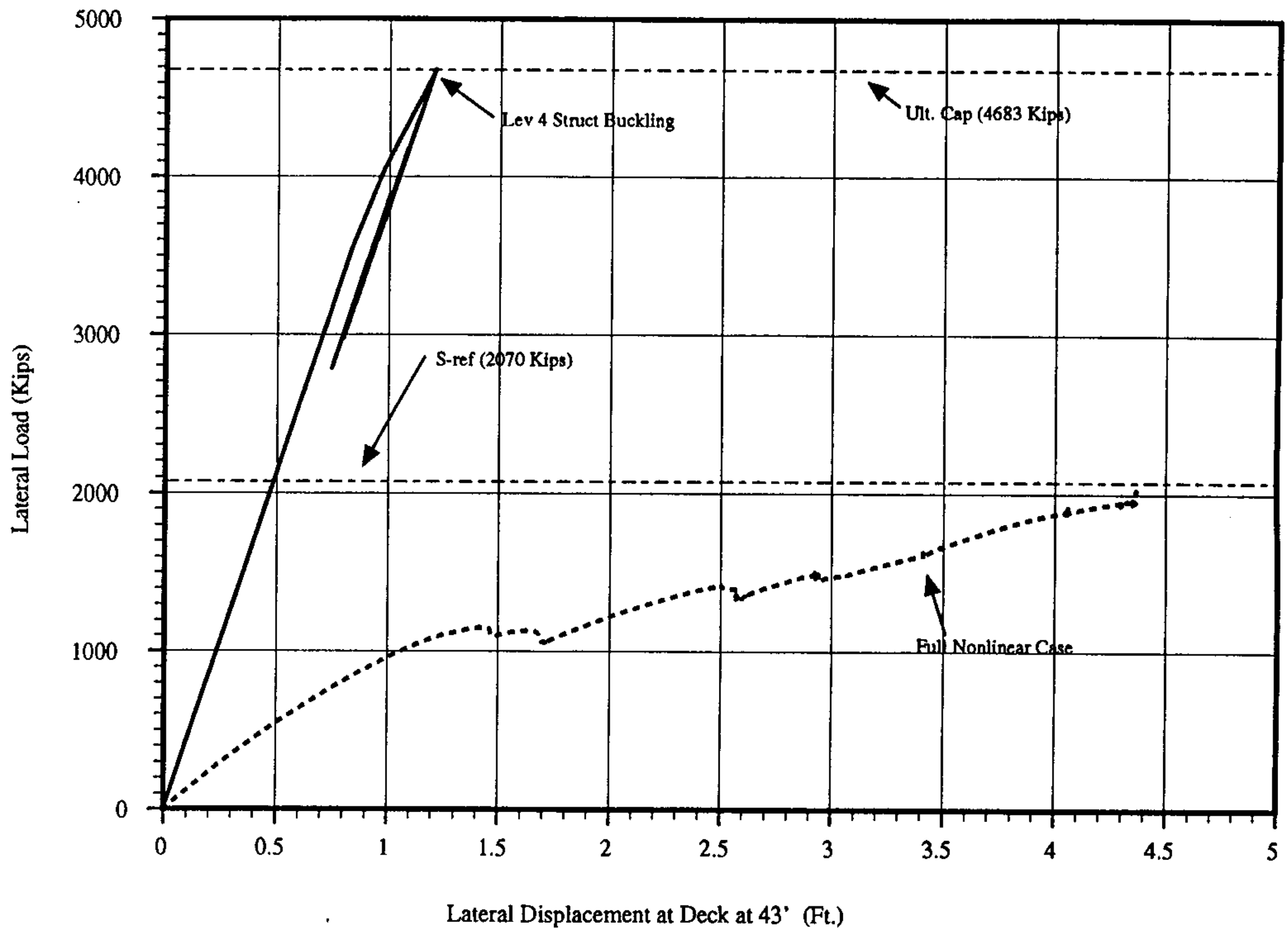


FIGURE A.5.2: Fixed Base Case Results - Diagonal Wave (270° TN)

PARTICIPANT "D" RESUBMITTAL

BENCHMARK ANALYSIS

TRIAL APPLICATION OF THE DRAFT API RP2A-WSD PROCEDURES FOR ASSESSMENT OF EXISTING PLATFORMS

JOINT INDUSTRY PROJECT

Summary

An ultimate strength analysis was performed on a 4 pile platform located in Ship Shoal Block 224 of the Gulf of Mexico. This platform was selected as a benchmark platform to be analyzed by several different organizations as part of a joint industry project sponsored by the MMS and managed by PMB Engineering, Inc. These analyses are to be compared to determine the variability of the ultimate strength results and to evaluate the use and application of the API draft guidelines on the assessment of existing platforms.

The results of this ultimate strength analysis showed that the benchmark platform did not pass the assessment process for the designated exposure category of a manned, evacuated platform with significant environmental impact. The minimum reserve strength ratio (for a diagonal approach) was 0.92 and the ultimate lateral load capacity was 2200 kips or 71% of the lateral load required by the ultimate strength assessment criteria for the Gulf of Mexico. The governing failure mechanism was the ultimate capacity of the pile foundation.

Part A: Benchmark Analysis

A.1 Environmental Criteria

Three wave approach directions were used with the following environmental criteria for each direction:

1. Approach angle 315 degree True
   Wave height = 64.6 feet
   Wave period = 13.5 sec.
   Current Profile = 2.3 knots @ 275 deg. True
   Storm Surge = 3.5 feet
   Wind Speed = 85 knots @ 10 meters above msl
   Wave load in Deck = 142 kips based on 64.6 foot wave

2. Approach angle 225 degree True
   Wave height = 61.2 feet
   Wave period = 13.5 sec.
   Current Profile = 2.3 knots @ 275 deg. True
   Storm Surge = 3.5 feet
   Wind Speed = 85 knots @ 10 meters above msl
   Wave load in Deck = 91 kips based on 61.2 foot wave

3. Approach angle 270 degree True
Wave height = 68.0 feet
Wave period = 13.5 sec.
Current Profile = 2.3 knots @ 275 deg. True
Storm Surge = 3.5 feet
Wind Speed = 85 knots @ 10 meters above msl
Wave load in Deck = 450 kips based on 68.0 foot wave

The wave and current directions were based on API RP2A 20th edition with the wave heights for each direction calculated based on the factors given in RP2A figure 2.3.4-4. Because the waves generated forces on the deck, the platform wave and current loadings were calculated separately for the jacket and the deck. The wave loads in the deck were calculated with the procedure given in section C17.6 of the API RP2A draft section 17. The total platform loading was calculated as the sum of the jacket and deck loadings.

The 20th edition RP2A recipe was used for calculating the wave and current loading below the +33 foot elevation, with the following criteria:

- Wave kinematics factor:	0.88
- Current blockage:	0.80 end-on/broadside
	0.85 diagonal
- Marine Growth:	1.5" (+1 ft. to mudline)
- Drag Coefficient:	0.65 (smooth)
	1.05 (rough)
- Inertia Coefficient:	1.60 (smooth)
	1.20 (rough)
- Wave theory:	Stokes 5th
- Conductor Shielding:	neglected
- Apparent wave period:	13.5 - 14.4 sec

A.1.1 API RP2A 20th ed. Environmental criteria

For use in the calculation of the platform reserve strength ratio, the lateral loads for the 100 year loading based on the RP2A 20th ed. criteria were determined. These criteria are listed below:

a. Wave height (max):	63 feet
b. Storm Tide:	3.5 feet
c. Deck min. height:	49.5 feet
d. Wave and Current direction:	Fig. 2.3.4-4 & 2.3.4-5
e. Current Speed:	2.1 knots
f. Wave Period:	13.0 sec.
g. Wind Speed:	80 knots

The associated wave height and current for various approach angles are as follows:

Approach Angle w.r.t Str. North/True North	Height (ft)	Current (kts)	Current Angle (True)
0 / 315	59.9	2.1	275
45 / 0	53.6	2.1	275
90 / 45	44.1	1.6	320
135 / 90	44.1	0.3	50
180 / 135	44.1	1.4	215
225 / 180	47.2	2.1	265
270 / 225	56.7	2.1	275
315 / 270	63.0	2.1	275

The lateral loads (kips) for each wave approach angle are listed below:

Approach Angle w.r.t. Str. North / True North	Jacket Force	Deck Force	Base Shear Total
0 / 315	1914	80	1994
45 / 0	1342	21	1363
90 / 45	845	0	845
135 / 90	681	0	681
180 / 135	884	0	884
225 / 180	1128	0	1128
270 / 225	1803	47	1850
315 / 270	2269	133	2402

A.2 3-D Model Generation

The structure model was developed based on the drawings and information given in the Design Basis Document including the 3 revisions issued. The model was generated with the SACS program by Engineering Dynamics, Inc. and converted for use in the nonlinear structural analysis program, Capacity Analysis Program (CAP) by PMB Engineering, Inc. The SACS program was used initially to perform a design level analysis for comparison with the ultimate strength analysis. The model (as shown in Figure A.2.1) included the jacket, deck, piles, conductors and appurtenances such as boat landings and risers.

The ultimate strength analysis utilized the following elements for member modeling:

Legs/Piles	Beam Column	(refer to CAP documentation)
Vertical Diagonal Braces	Marshal Strut	"
Horizontal Braces	Linear Beam	"
Deck Members	Linear Beam	"

The major structural joints were checked with the SACS program for the design level analysis. These joints were determined to have acceptable stress ratios for the design level loading and were therefore considered to be adequate to transfer the ultimate strength loads.

The soil springs were generated by the CAP program based on the parameters given in the Design Basis Document. The ultimate capacity of the piles was calculated by the CAP program and checked with hand calculations.

The material yield strength for all structural members (jacket, deck and foundation) was 42 ksi.

A.3 Software Description

The PMB Capacity Analysis Program (CAP) was used and is described in detail in the PMB documentation.

A.4 Ultimate Strength Analysis Results

The results of the ultimate strength analyses are summarized for each approach direction in the following Figures and Tables:

Figure A.4.1	0 degree approach ultimate lateral load vs. deck deflection.
Figure A.4.2	270 degree approach ultimate lateral load vs. deck deflection.
Figure A.4.3	315 degree approach ultimate lateral load vs. deck deflection.
Table A.4.1	0 degree approach tabulated results for load step, displacement and load.
Table A.4.2	270 degree approach tabulated results for load step, displacement and load.
Table A.4.3	315 degree approach tabulated results for load step, displacement and load.
Table A.4.4	315 degree approach tabulated failure modes and inelastic events.
Figures A.4.4	Governing pile capacity curve for Pile A2 for 315 degree approach angle.
Figures A.4.5	Governing pile capacity curve for Pile B1 for 315 degree approach angle.
Figure A.4.6	Deflected shape for 315 degree approach angle.

A.5 Design Level Analysis

A design level analysis is not applicable for this benchmark platform because of the inadequate deck height.

FIGURE A.4.1 – 315 DEGREE TRUE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1410 kips

100 year, 20th ed. ref. load (Sref) = 1994 kips

Ultimate Strength Analysis Load (Susa) = 2429 kips

Ultimate Capacity (Ru) = 2764 kips

Reserve Strength Ratio = 1.4

Platform Failure Mode = Foundation

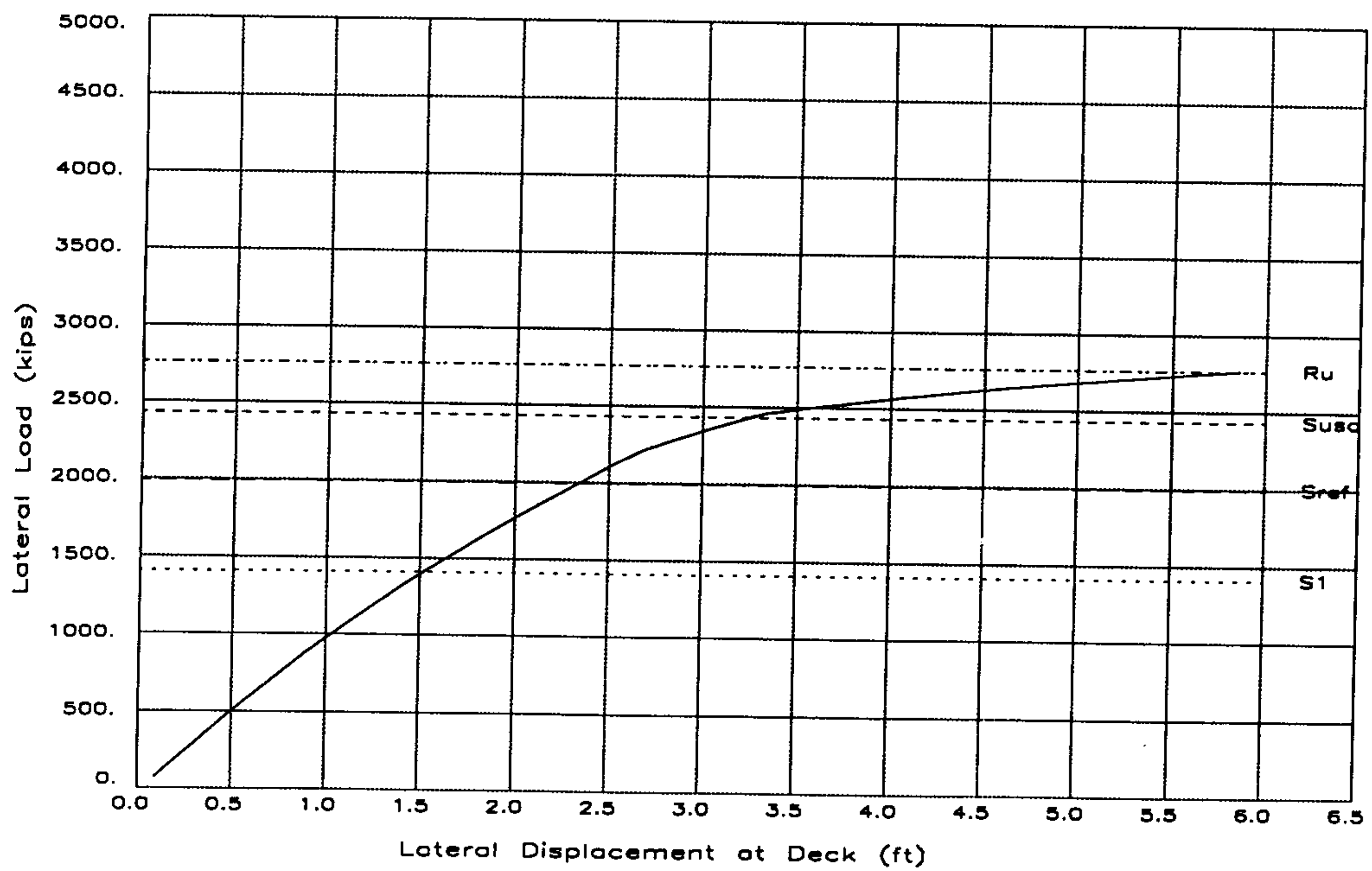


FIGURE A.4.2 – 225 DEGREE TRUE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1375 kips

100 year, 20th ed. ref. load (Sref) = 1850 kips

Ultimate Strength Analysis Load (Susa) = 2246 kips

Ultimate Capacity (Ru) = 2623 kips

Reserve Strength Ratio = 1.4

Platform Failure Mode = Foundation

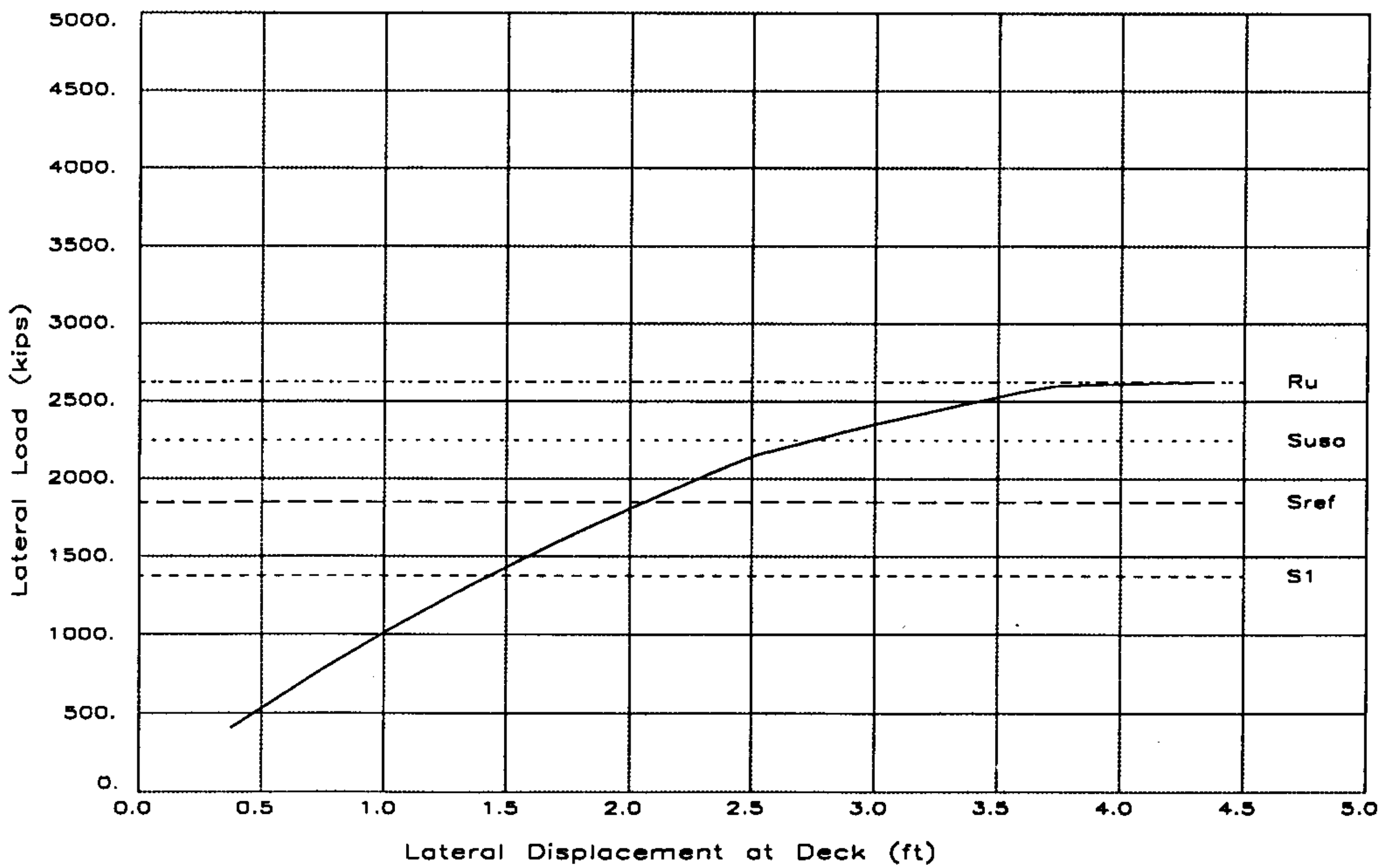


FIGURE A.4.3 – 270 DEGREE TRUE DIRECTION

Load at 1st component I.R.=1.0 (S1) = 1310 kips

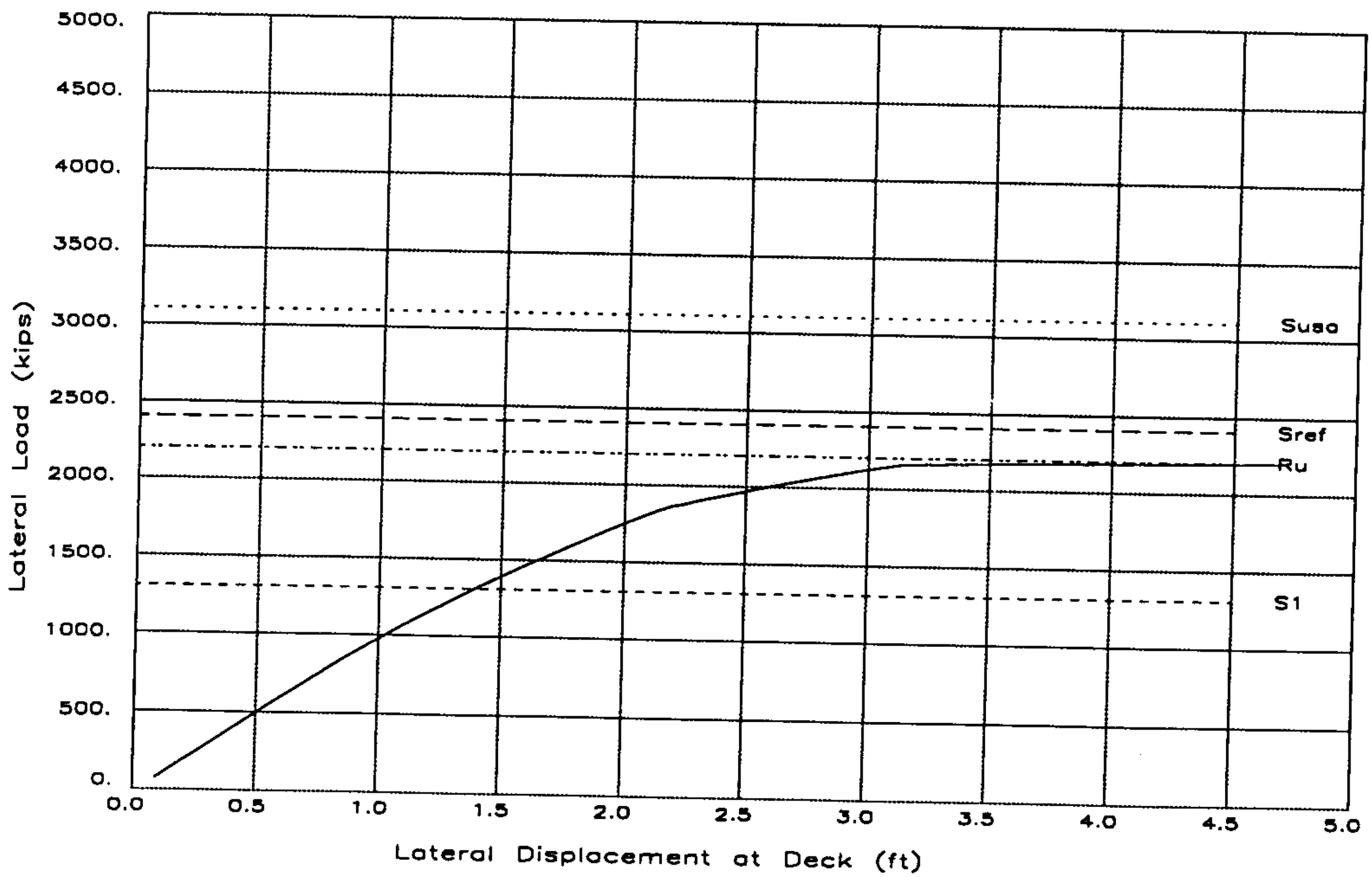
100 year, 20th ed. ref. load (Sref) = 2402 kips

Ultimate Strength Analysis Load (Susa) = 3108 kips

Ultimate Capacity (Ru) = 2200 kips

Reserve Strength Ratio = .92

Platform Failure Mode = Foundation



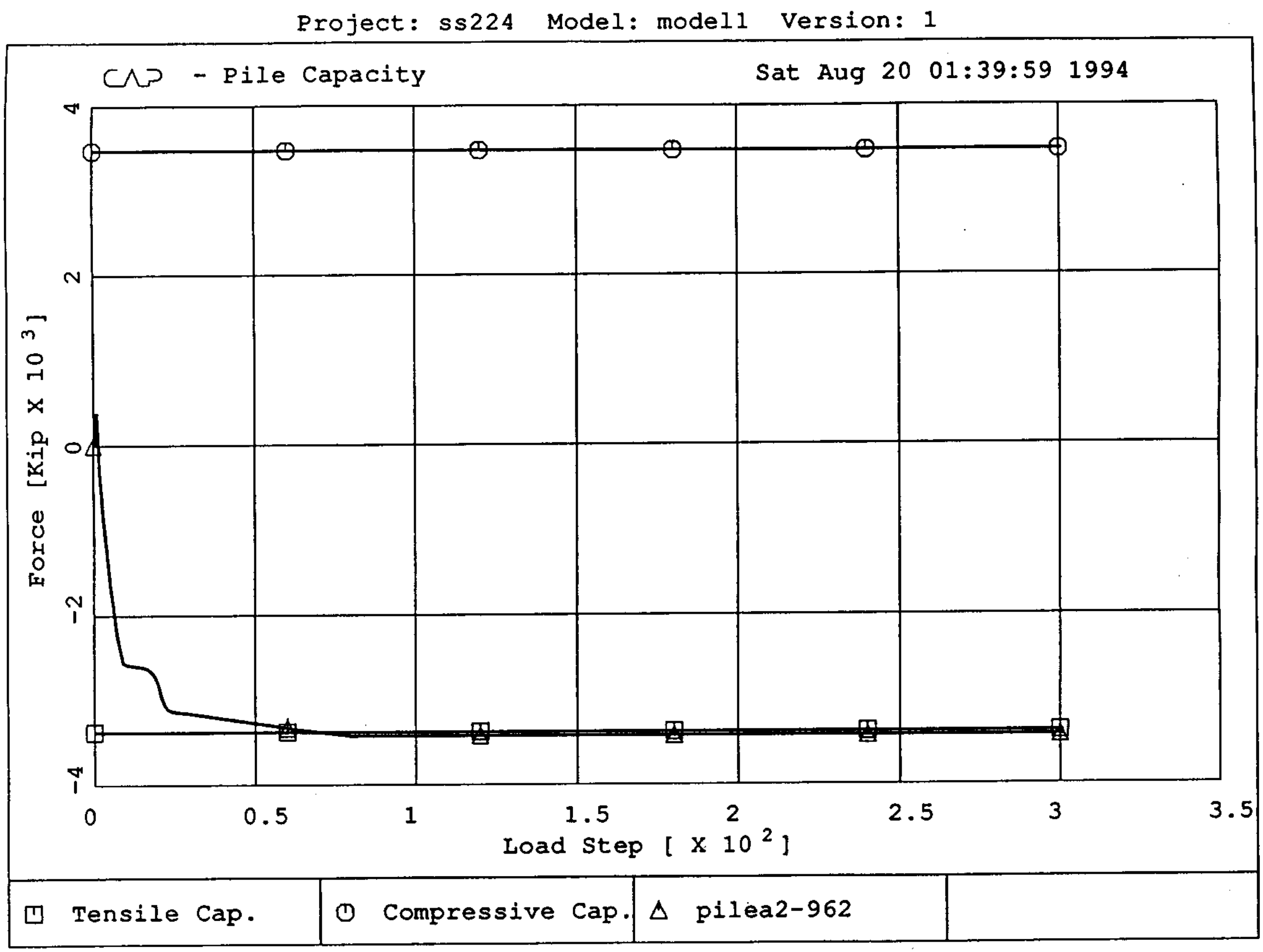


Figure A.4.4 135 deg. pile cap.

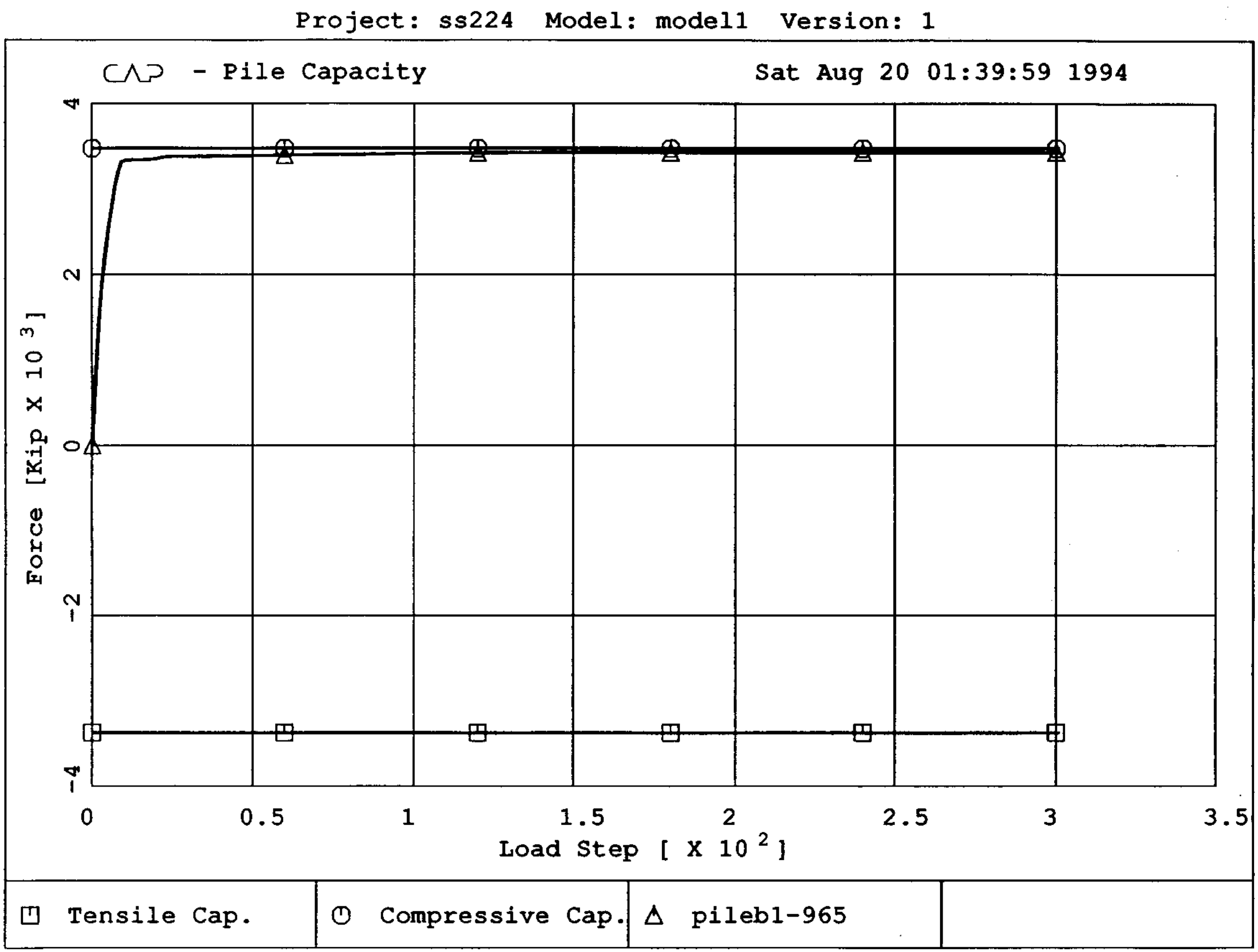


Figure A.4.5 135 deg. pile cap.

Project: ss224 Model: modell Version: 1   Sat Aug 20 09:59:24 1994 *

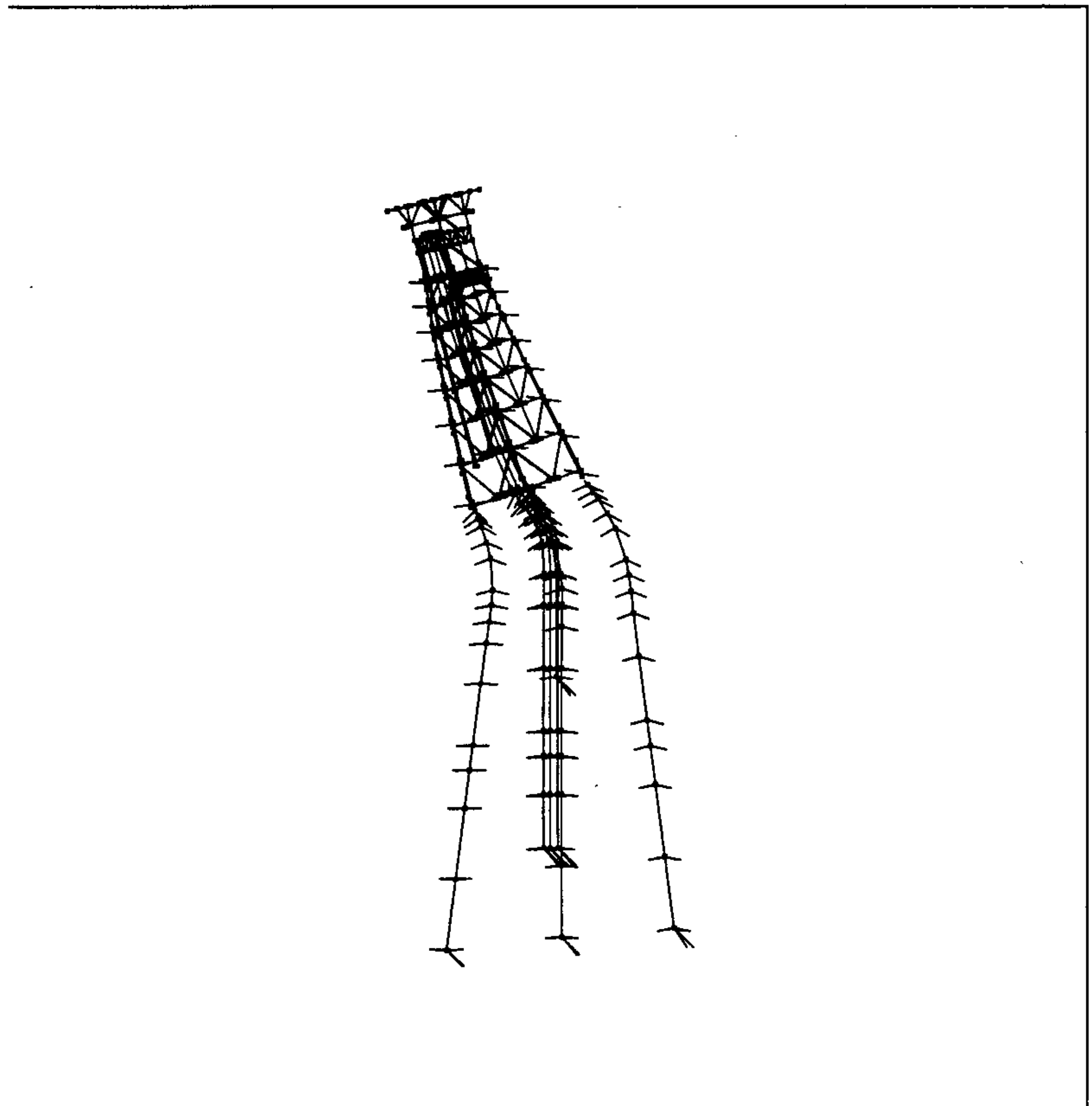


Figure A.4.6 Deflected Shape at 315 deg.

** TABLE A.4.1 0 DEGREE DIRECTION

CAP Results Table Sun Aug 21 01:14:30 1994

Project: ss224 Model: modell Version: 1

Cut Plane Force Fx Kip

0 degree direction

Load Step	X Deck Disp Ft	Force Kips
1	0.090	69.778
2	0.490	493.597
3	0.886	873.763
4	1.187	1138.643
5	1.419	1333.144
6	1.616	1487.128
7	1.858	1666.409
8	2.045	1797.934
9	2.191	1895.538
10	2.370	2014.628
11	2.447	2065.929
12	2.524	2116.805
13	2.602	2163.784
14	2.645	2188.502
15	2.664	2199.138
16	2.696	2217.536
17	2.699	2219.121
18	2.701	2219.877
19	2.705	2221.273
20	2.711	2223.689
21	2.721	2227.875
22	2.740	2235.125
23	2.771	2247.682
24	2.826	2269.431
25	2.921	2306.752
26	3.037	2351.218
27	3.239	2425.838
28	3.266	2435.104
29	3.313	2451.167
30	3.333	2458.000
31	3.335	2458.739
32	3.339	2459.854
33	3.341	2460.203
34	3.346	2460.907
35	3.353	2462.126
36	3.366	2464.230
37	3.389	2467.877
38	3.429	2474.127
39	3.497	2484.652
40	3.616	2503.509
41	3.760	2526.402
42	4.012	2564.079
43	4.075	2572.430
44	4.102	2575.992
45	4.114	2577.534
46	4.134	2580.234
47	4.143	2581.381
48	4.147	2581.878
49	4.154	2582.739
50	4.160	2583.641
51	4.163	2584.021
52	4.168	2584.656
53	4.175	2585.496
54	4.187	2587.062
55	4.192	2587.731
56	4.201	2588.901
57	4.205	2589.401
58	4.211	2590.271
59	4.218	2591.174
60	4.221	2591.560
61	4.226	2592.227
62	4.233	2593.107
63	4.240	2593.976
64	4.247	2594.865
65	4.250	2595.250
66	4.255	2595.917
67	4.262	2596.801
68	4.269	2597.684
69	4.275	2598.571
70	4.282	2599.455
71	4.289	2600.338
72	4.296	2601.221
73	4.303	2602.103
74	4.309	2602.986
75	4.316	2603.868
76	4.323	2604.751
77	4.330	2605.633
78	4.337	2606.515
79	4.343	2607.397
80	4.350	2608.279
81	4.357	2609.161
82	4.364	2610.043
83	4.371	2610.922
84	4.378	2611.802
85	4.384	2612.680
86	4.391	2613.560
87	4.398	2614.438
88	4.405	2615.313
89	4.412	2616.186
90	4.418	2617.056
91	4.425	2617.926
92	4.432	2618.795
93	4.439	2619.663
94	4.445	2620.532
95	4.452	2621.401
96	4.459	2622.269
97	4.466	2623.137
98	4.473	2624.007
99	4.479	2624.870
100	4.486	2625.733
101	4.493	2626.592
102	4.500	2627.458
103	4.506	2628.325
104	4.513	2629.197
105	4.520	2630.052
106	4.527	2630.905
107	4.533	2631.756
108	4.540	2632.612
109	4.547	2633.480
110	4.554	2634.335
111	4.561	2635.200
112	4.567	2636.050
113	4.574	2636.890
114	4.581	2637.751
115	4.588	2638.623
116	4.594	2639.441
117	4.601	2640.154
118	4.608	2640.860
119	4.615	2641.558

120	4.622	2642.254
121	4.629	2642.948
122	4.635	2643.642
123	4.642	2644.334
124	4.649	2645.026
125	4.656	2645.718
126	4.663	2646.410
127	4.670	2647.102
128	4.676	2647.794
129	4.683	2648.486
130	4.690	2649.178
131	4.697	2649.869
132	4.704	2650.562
133	4.710	2651.253
134	4.717	2651.942
135	4.724	2652.630
136	4.731	2653.318
137	4.738	2654.006
138	4.745	2654.695
139	4.751	2655.383
140	4.758	2656.071
141	4.765	2656.759
142	4.772	2657.447
143	4.779	2658.135
144	4.786	2658.823
145	4.792	2659.511
146	4.799	2660.199
147	4.806	2660.888
148	4.813	2661.576
149	4.820	2662.264
150	4.826	2662.952
151	4.833	2663.640
152	4.840	2664.328
153	4.847	2665.016
154	4.854	2665.705
155	4.861	2666.393
156	4.867	2667.081
157	4.874	2667.769
158	4.881	2668.453
159	4.888	2669.135
160	4.895	2669.818
161	4.902	2670.500
162	4.908	2671.182
163	4.915	2671.865
164	4.922	2672.547
165	4.929	2673.229
166	4.936	2673.912
167	4.943	2674.594
168	4.949	2675.276
169	4.956	2675.959
170	4.963	2676.641
171	4.970	2677.323
172	4.977	2678.006
173	4.984	2678.688
174	4.990	2679.370
175	4.997	2680.052
176	5.004	2680.735
177	5.011	2681.417
178	5.018	2682.099
179	5.024	2682.782
180	5.031	2683.463
181	5.038	2684.145
182	5.045	2684.826
183	5.052	2685.508
184	5.059	2686.189
185	5.065	2686.871
186	5.072	2687.553
187	5.079	2688.234
188	5.086	2688.916
189	5.093	2689.597
190	5.100	2690.279
191	5.106	2690.960
192	5.113	2691.642
193	5.120	2692.324
194	5.127	2693.005
195	5.134	2693.687
196	5.141	2694.368
197	5.147	2695.050
198	5.154	2695.731
199	5.161	2696.413
200	5.168	2697.095
201	5.175	2697.776
202	5.182	2698.458
203	5.188	2699.139
204	5.195	2699.821
205	5.202	2700.502
206	5.209	2701.184
207	5.216	2701.865
208	5.223	2702.546
209	5.229	2703.228
210	5.236	2703.910
211	5.243	2704.591
212	5.250	2705.272
213	5.257	2705.951
214	5.263	2706.631
215	5.270	2707.311
216	5.277	2707.991
217	5.284	2708.671
218	5.291	2709.349
219	5.298	2710.028
220	5.304	2710.706
221	5.311	2711.384
222	5.318	2712.063
223	5.325	2712.741
224	5.332	2713.419
225	5.339	2714.098
226	5.345	2714.776
227	5.352	2715.454
228	5.359	2716.133
229	5.366	2716.811
230	5.373	2717.489
231	5.379	2718.167
232	5.386	2718.845
233	5.393	2719.524
234	5.400	2720.202
235	5.407	2720.880
236	5.414	2721.558
237	5.420	2722.236
238	5.427	2722.914
239	5.434	2723.592
240	5.441	2724.269
241	5.448	2724.947
242	5.454	2725.625
243	5.461	2726.303
244	5.468	2726.978
245	5.475	2727.649
246	5.482	2728.320
247	5.489	2728.992
248	5.495	2729.663
249	5.502	2730.335
250	5.509	2731.006
251	5.516	2731.677

252	5.523	2732.346
253	5.529	2733.010
254	5.536	2733.674
255	5.543	2734.338
256	5.550	2735.002
257	5.557	2735.666
258	5.564	2736.330
259	5.570	2736.994
260	5.577	2737.657
261	5.584	2738.321
262	5.591	2738.985
263	5.598	2739.649
264	5.604	2740.313
265	5.611	2740.974
266	5.618	2741.622
267	5.625	2742.273
268	5.632	2742.924
269	5.638	2743.576
270	5.645	2744.227
271	5.652	2744.878
272	5.659	2745.530
273	5.666	2746.181
274	5.672	2746.832
275	5.679	2747.483
276	5.686	2748.135
277	5.693	2748.786
278	5.700	2749.438
279	5.707	2750.088
280	5.713	2750.740
281	5.720	2751.390
282	5.727	2752.042
283	5.734	2752.692
284	5.741	2753.344
285	5.747	2753.994
286	5.754	2754.646
287	5.761	2755.296
288	5.768	2755.948
289	5.775	2756.599
290	5.781	2757.250
291	5.788	2757.901
292	5.795	2758.551
293	5.802	2759.202
294	5.809	2759.853
295	5.816	2760.503
296	5.822	2761.154
297	5.829	2761.804
298	5.836	2762.454
299	5.843	2763.102
300	5.850	2763.748
301	5.856	2764.394

*** TABLE A.4.2 ***

CAP Results Table Mon Aug 22 04:44:43 1994

Project: ss224 Model: modell Version: 1

Cut Plane Force Fy - Kip

270 deg. direction

Load Step	Y Deck Disp Ft	Force Kips
2	0.375	404.204
3	0.746	777.063
4	1.029	1036.981
5	1.309	1280.208
6	1.550	1471.852
7	1.717	1599.117
8	1.862	1707.274
9	2.040	1832.817
10	2.218	1956.301
11	2.397	2076.448
12	2.514	2148.889
13	2.523	2153.627
14	2.528	2155.584
15	2.536	2159.071
16	2.550	2165.111
17	2.575	2175.565
18	2.617	2193.606
19	2.691	2223.755
20	2.754	2249.710
21	2.864	2294.373
22	3.055	2368.972
23	3.089	2381.284
24	3.147	2402.595
25	3.153	2404.899
26	3.164	2408.883
27	3.169	2410.545
28	3.174	2412.518
29	3.180	2414.124
30	3.185	2415.547
31	3.190	2417.372
32	3.196	2419.332
33	3.205	2422.859
34	3.209	2424.376
35	3.211	2425.029
36	3.214	2426.139
37	3.220	2428.150
38	3.222	2429.028
39	3.226	2430.550
40	3.228	2431.205
41	3.231	2432.324
42	3.237	2434.237
43	3.242	2436.119
44	3.248	2438.121
45	3.253	2440.118
46	3.259	2442.090
47	3.264	2444.036
48	3.270	2445.959
49	3.275	2447.871
50	3.281	2449.775
51	3.286	2451.676
52	3.292	2453.573
53	3.297	2455.469
54	3.303	2457.366
55	3.308	2459.262
56	3.313	2461.158
57	3.319	2463.053
58	3.324	2464.945
59	3.330	2466.833
60	3.335	2468.722
61	3.341	2470.608
62	3.346	2472.489
63	3.352	2474.370
64	3.357	2476.252
65	3.363	2478.126
66	3.368	2479.999
67	3.373	2481.872
68	3.379	2483.745
69	3.384	2485.617
70	3.390	2487.490
71	3.395	2489.361
72	3.401	2491.234
73	3.406	2493.106
74	3.412	2494.978
75	3.417	2496.850
76	3.423	2498.721
77	3.428	2500.593
78	3.433	2502.465
79	3.439	2504.336
80	3.444	2506.208
81	3.450	2508.079
82	3.455	2509.944
83	3.461	2511.805
84	3.466	2513.667
85	3.472	2515.527
86	3.477	2517.388
87	3.482	2519.248
88	3.488	2521.109
89	3.493	2522.969
90	3.499	2524.830
91	3.504	2526.690
92	3.510	2528.551
93	3.515	2530.411
94	3.521	2532.272
95	3.526	2534.132
96	3.531	2535.992
97	3.537	2537.858
98	3.542	2539.704
99	3.548	2541.529
100	3.553	2543.344
101	3.558	2545.104
102	3.564	2546.852
103	3.569	2548.606
104	3.574	2550.420
105	3.580	2552.265
106	3.585	2554.096
107	3.591	2555.889
108	3.596	2557.611
109	3.601	2559.127
110	3.607	2560.676
111	3.612	2562.222
112	3.617	2563.769
113	3.623	2565.314
114	3.628	2566.860
115	3.633	2568.402
116	3.638	2569.942
117	3.644	2571.483
118	3.649	2573.024
119	3.654	2574.564
120	3.660	2576.104

121	3.665	2577.645
122	3.670	2579.186
123	3.676	2580.726
124	3.681	2582.267
125	3.686	2583.784
126	3.692	2585.250
127	3.697	2586.734
128	3.702	2588.219
129	3.708	2589.704
130	3.713	2591.189
131	3.719	2592.673
132	3.724	2594.158
133	3.730	2595.642
134	3.735	2597.042
135	3.740	2597.212
136	3.745	2597.559
137	3.750	2597.889
138	3.755	2598.221
139	3.760	2598.554
140	3.764	2598.887
141	3.769	2599.181
142	3.774	2599.381
143	3.779	2599.581
144	3.784	2599.780
145	3.786	2599.867
146	3.791	2600.065
147	3.796	2600.262
148	3.816	2601.057
149	3.824	2601.404
150	3.828	2601.555
151	3.829	2601.620
152	3.834	2601.817
153	3.836	2601.904
154	3.840	2602.054
155	3.845	2602.250
156	3.847	2602.338
157	3.851	2602.487
158	3.856	2602.684
159	3.858	2602.771
160	3.862	2602.921
161	3.866	2603.117
162	3.869	2603.205
163	3.874	2603.401
164	3.876	2603.489
165	3.879	2603.638
166	3.884	2603.834
167	3.886	2603.922
168	3.891	2604.118
169	3.893	2604.206
170	3.897	2604.355
171	3.902	2604.551
172	3.904	2604.639
173	3.909	2604.835
174	3.911	2604.923
175	3.915	2605.072
176	3.920	2605.268
177	3.922	2605.355
178	3.927	2605.553
179	3.932	2605.750
180	3.934	2605.837
181	3.937	2605.985
182	3.939	2606.051
183	3.942	2606.162
184	3.947	2606.358
185	3.949	2606.445
186	3.954	2606.641
187	3.956	2606.728
188	3.961	2606.923
189	3.966	2607.120
190	3.971	2607.318
191	3.975	2607.515
192	3.980	2607.712
193	3.985	2607.910
194	3.990	2608.106
195	3.999	2608.449
196	4.002	2608.600
197	4.009	2608.857
198	4.011	2608.971
199	4.016	2609.165
200	4.021	2609.360
201	4.023	2609.447
202	4.028	2609.642
203	4.030	2609.728
204	4.034	2609.876
205	4.039	2610.070
206	4.041	2610.156
207	4.046	2610.351
208	4.048	2610.437
209	4.052	2610.585
210	4.057	2610.779
211	4.059	2610.865
212	4.064	2611.060
213	4.066	2611.146
214	4.069	2611.294
215	4.074	2611.488
216	4.076	2611.575
217	4.081	2611.769
218	4.083	2611.855
219	4.087	2612.003
220	4.092	2612.197
221	4.094	2612.283
222	4.099	2612.477
223	4.101	2612.563
224	4.106	2612.758
225	4.111	2612.952
226	4.113	2613.038
227	4.117	2613.185
228	4.118	2613.250
229	4.121	2613.360
230	4.126	2613.552
231	4.131	2613.745
232	4.133	2613.832
233	4.138	2614.026
234	4.140	2614.112
235	4.143	2614.260
236	4.148	2614.453
237	4.150	2614.540
238	4.155	2614.734
239	4.158	2614.820
240	4.161	2614.968
241	4.166	2615.161
242	4.168	2615.248
243	4.173	2615.442
244	4.175	2615.528
245	4.179	2615.676
246	4.184	2615.869
247	4.186	2615.956
248	4.191	2616.149
249	4.193	2616.236
250	4.198	2616.430
251	4.203	2616.624
252	4.205	2616.710

253	4.208	2616.857
254	4.210	2616.922
255	4.213	2617.032
256	4.218	2617.226
257	4.220	2617.312
258	4.225	2617.506
259	4.227	2617.592
260	4.231	2617.739
261	4.235	2617.933
262	4.238	2618.019
263	4.242	2618.213
264	4.245	2618.299
265	4.249	2618.494
266	4.254	2618.687
267	4.256	2618.772
268	4.260	2618.918
269	4.262	2618.983
270	4.267	2619.176
271	4.269	2619.261
272	4.274	2619.454
273	4.276	2619.540
274	4.281	2619.733
275	4.283	2619.819
276	4.288	2620.013
277	4.293	2620.206
278	4.295	2620.292
279	4.298	2620.438
280	4.300	2620.502
281	4.303	2620.612
282	4.308	2620.804
283	4.310	2620.891
284	4.315	2621.083
285	4.317	2621.169
286	4.320	2621.316
287	4.325	2621.508
288	4.327	2621.594
289	4.332	2621.787
290	4.334	2621.873
291	4.338	2622.020
292	4.343	2622.212
293	4.345	2622.298
294	4.350	2622.491
295	4.352	2622.577
296	4.356	2622.724
297	4.361	2622.916
298	4.363	2623.002
299	4.368	2623.196
300	4.373	2623.389
301	4.375	2623.475

** TABLE A.4.3 315 DEG. APPROACH ***

CAP Results Table Sat Aug 20 01:39:59 1994

Project: ss224 Model: model1 Version: 1

Cut Plane Force Fx - Kip

315 deg. direction

Load Step	X Deck Disp Ft	Force Kips
1	0.090	72.338
2	0.534	533.353
3	0.873	865.383
4	1.133	1097.983
5	1.394	1310.372
6	1.596	1462.651
7	1.844	1639.369
8	2.006	1750.900
9	2.148	1839.469
10	2.166	1850.084
11	2.174	1854.683
12	2.178	1856.656
13	2.184	1859.376
14	2.187	1860.549
15	2.193	1862.651
16	2.204	1866.292
17	2.222	1872.600
18	2.253	1883.524
19	2.306	1902.445
20	2.400	~~1935.116~~
21	2.561	1989.089
22	2.676	~~2025.554~~
23	2.726	2041.305
24	2.747	2048.096
25	2.757	2051.036
26	2.761	2052.298
27	2.768	2054.499
28	2.771	2055.446
29	2.776	2057.052
30	2.783	2059.266
31	2.790	2061.397
32	2.797	2063.511
33	2.805	2065.613
34	2.812	2067.712
35	2.819	2069.809
36	2.826	2071.905
37	2.833	2074.000
38	2.840	2076.094
39	2.847	2078.189
40	2.854	2080.283
41	2.862	2082.378
42	2.869	2084.473
43	2.876	2086.567
44	2.883	2088.662
45	2.890	2090.757
46	2.897	2092.851
47	2.904	2094.946
48	2.911	2097.040
49	2.918	2099.135
50	2.926	2101.229
51	2.933	2103.323
52	2.940	2105.415
53	2.947	2107.507
54	2.954	2109.597
55	2.961	2111.687
56	2.968	2113.776
57	2.975	2115.866
58	2.983	2117.955
59	2.990	2120.045
60	2.997	2122.134
61	3.004	2124.223
62	3.011	2126.252
63	3.018	2127.972
64	3.026	2129.730
65	3.033	2131.488
66	3.040	2133.246
67	3.047	2135.004
68	3.054	2136.761
69	3.062	2138.517
70	3.069	2140.272
71	3.076	2142.027
72	3.083	2143.782
73	3.091	2145.538
74	3.098	2147.293
75	3.105	2149.049
76	3.112	2150.804
77	3.120	2152.559
78	3.127	2154.312
79	3.134	2156.065
80	3.141	2157.795
81	3.149	2158.118
82	3.157	2158.435
83	3.164	2158.752
84	3.172	2159.077
85	3.185	2159.633
86	3.209	2160.596
87	3.211	2160.679
88	3.215	2160.859
89	3.223	2161.164
90	3.231	2161.481
91	3.238	2161.799
92	3.248	2162.197
93	3.264	2162.878
94	3.293	2164.057
95	3.296	2164.159
96	3.301	2164.379
97	3.303	2164.471
98	3.307	2164.637
99	3.314	2164.917
100	3.322	2165.242
101	3.325	2165.376
102	3.331	2165.616
103	3.333	2165.716
104	3.338	2165.896
105	3.345	2166.201
106	3.353	2166.526
107	3.356	2166.659
108	3.362	2166.899
109	3.365	2166.999
110	3.369	2167.180
111	3.377	2167.484
112	3.384	2167.809
113	3.388	2167.942
114	3.393	2168.182
115	3.396	2168.282
116	3.400	2168.463
117	3.408	2168.767
118	3.415	2169.092
119	3.419	2169.225

120	3.425	2169.466
121	3.427	2169.565
122	3.431	2169.746
123	3.439	2170.051
124	3.447	2170.375
125	3.450	2170.508
126	3.456	2170.749
127	3.458	2170.849
128	3.463	2171.029
129	3.470	2171.333
130	3.478	2171.658
131	3.481	2171.791
132	3.487	2172.032
133	3.489	2172.132
134	3.494	2172.312
135	3.501	2172.617
136	3.509	2172.941
137	3.512	2173.074
138	3.518	2173.313
139	3.521	2173.413
140	3.525	2173.592
141	3.532	2173.896
142	3.540	2174.219
143	3.544	2174.352
144	3.549	2174.592
145	3.552	2174.691
146	3.556	2174.871
147	3.564	2175.121
148	3.595	2175.880
149	3.648	2177.164
150	3.672	2177.680
151	3.682	2177.904
152	3.699	2178.321
153	3.707	2178.488
154	3.720	2178.801
155	3.725	2178.927
156	3.735	2179.161
157	3.739	2179.255
158	3.747	2179.431
159	3.759	2179.736
160	3.765	2179.858
161	3.775	2180.087
162	3.779	2180.179
163	3.786	2180.350
164	3.798	2180.647
165	3.804	2180.766
166	3.813	2180.989
167	3.817	2181.078
168	3.824	2181.245
169	3.836	2181.534
170	3.841	2181.650
171	3.850	2181.867
172	3.866	2182.242
173	3.873	2182.393
174	3.876	2182.458
175	3.881	2182.580
176	3.890	2182.791
177	3.905	2183.156
178	3.912	2183.303
179	3.923	2183.577
180	3.928	2183.687
181	3.937	2183.893
182	3.940	2183.975
183	3.947	2184.129
184	3.958	2184.396
185	3.963	2184.504
186	3.971	2184.704
187	3.986	2185.051
188	3.992	2185.190
189	4.003	2185.451
190	4.008	2185.555
191	4.016	2185.750
192	4.019	2185.829
193	4.025	2185.975
194	4.036	2186.228
195	4.041	2186.330
196	4.049	2186.520
197	4.062	2186.850
198	4.068	2186.982
199	4.079	2187.229
200	4.083	2187.328
201	4.091	2187.513
202	4.104	2187.834
203	4.110	2187.963
204	4.120	2188.203
205	4.125	2188.300
206	4.132	2188.480
207	4.145	2188.793
208	4.151	2188.919
209	4.161	2189.153
210	4.165	2189.247
211	4.172	2189.423
212	4.185	2189.728
213	4.191	2189.850
214	4.200	2190.078
215	4.204	2190.169
216	4.211	2190.340
217	4.215	2190.409
218	4.220	2190.537
219	4.229	2190.759
220	4.233	2190.848
221	4.240	2191.014
222	4.243	2191.081
223	4.249	2191.206
224	4.258	2191.422
225	4.262	2191.509
226	4.268	2191.671
227	4.271	2191.736
228	4.276	2191.857
229	4.285	2192.068
230	4.289	2192.153
231	4.296	2192.311
232	4.299	2192.374
233	4.304	2192.492
234	4.312	2192.698
235	4.316	2192.780
236	4.322	2192.934
237	4.325	2192.996
238	4.330	2193.111
239	4.338	2193.311
240	4.342	2193.391
241	4.348	2193.541
242	4.359	2193.800
243	4.364	2193.904
244	4.366	2193.949
245	4.369	2194.033
246	4.376	2194.179
247	4.386	2194.432
248	4.391	2194.533
249	4.399	2194.723
250	4.402	2194.799
251	4.408	2194.941

252	4.411	2194.998
253	4.415	2195.104
254	4.423	2195.289
255	4.426	2195.363
256	4.432	2195.501
257	4.442	2195.740
258	4.447	2195.836
259	4.454	2196.014
260	4.457	2196.086
261	4.463	2196.220
262	4.473	2196.452
263	4.477	2196.545
264	4.485	2196.719
265	4.488	2196.789
266	4.493	2196.919
267	4.496	2196.971
268	4.500	2197.069
269	4.507	2197.239
270	4.510	2197.307
271	4.515	2197.434
272	4.525	2197.654
273	4.529	2197.742
274	4.536	2197.907
275	4.539	2197.973
276	4.544	2198.096
277	4.553	2198.310
278	4.557	2198.396
279	4.564	2198.557
280	4.567	2198.621
281	4.572	2198.741
282	4.574	2198.790
283	4.578	2198.880
284	4.585	2199.036
285	4.587	2199.099
286	4.592	2199.216
287	4.595	2199.263
288	4.598	2199.351
289	4.605	2199.503
290	4.616	2199.766
291	4.621	2199.871
292	4.629	2200.069
293	4.633	2200.148
294	4.639	2200.296
295	4.642	2200.355
296	4.647	2200.457
297	4.654	2200.626
298	4.662	2200.812
299	4.675	2201.128
300	4.681	2201.254
301	4.691	2201.490

TAPLE A.4.4 - 315 DEG. APPROACH INELASTIC EVENT TABLE

? Inelastic Event Detailed Report Sat Aug 20
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›ject: ss224 Model: model1 Version: 1

. ›r Name	Load Step	Time	Element Type	Event Description
›2-60	21	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
›1-97	49	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
›1-97	54	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
›1-97	291	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
›1-97	292	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)

A.6 Fixed Base Ultimate Strength Analysis

An ultimate strength CAP analysis was performed on the benchmark platform with the pile elements below the mudline removed and the pile joints at the mudline pinned. The diagonal direction (315 deg.) was analyzed because this was the critical direction for the piled base model and because this direction produced the lowest lateral load required for 1st element yield in the jacket.

Results of the analysis are shown in Figure A.6.1 which shows the fixed base results together with the piled base results. The fixed base results are tabulated in Table A.6.1. Table A.6.2 lists the component failures and inelastic events for this analysis. The critical failure components were found to be the leg members which were modeled as beam columns and the diagonals which were modeled as Marshal Struts. The deflected shape of the fixed base model is shown in Figure A.6.2.

FIGURE A.6.1 – 270 DEG. TRUE DIRECTION FIXED & PILED BASE

100 year, 20th ed. ref. load (Sref) = 2402 kips

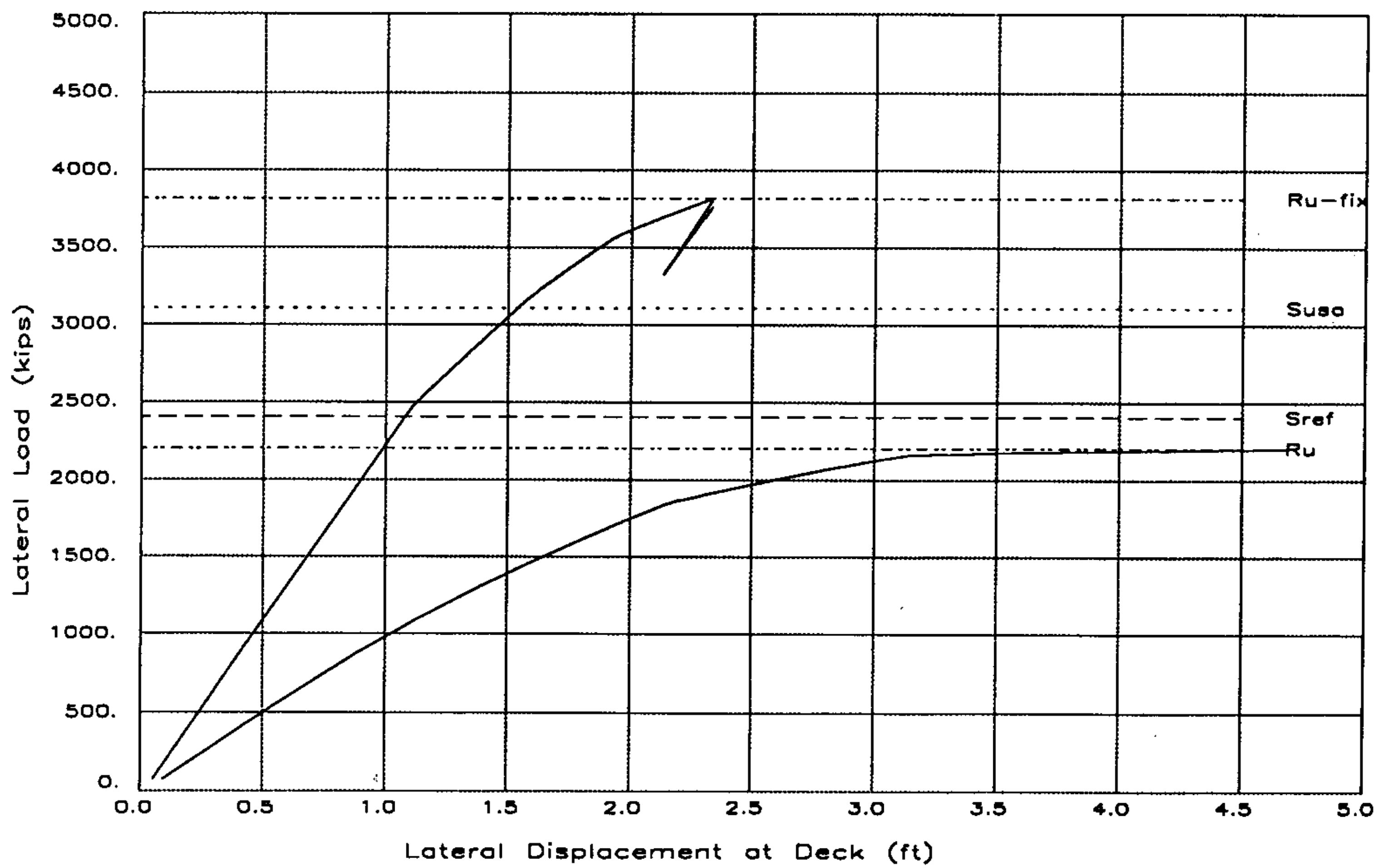
Ultimate Strength Analysis Load (Susa) = 3108 kips

Ultimate Capacity (Ru) = 2200 kips

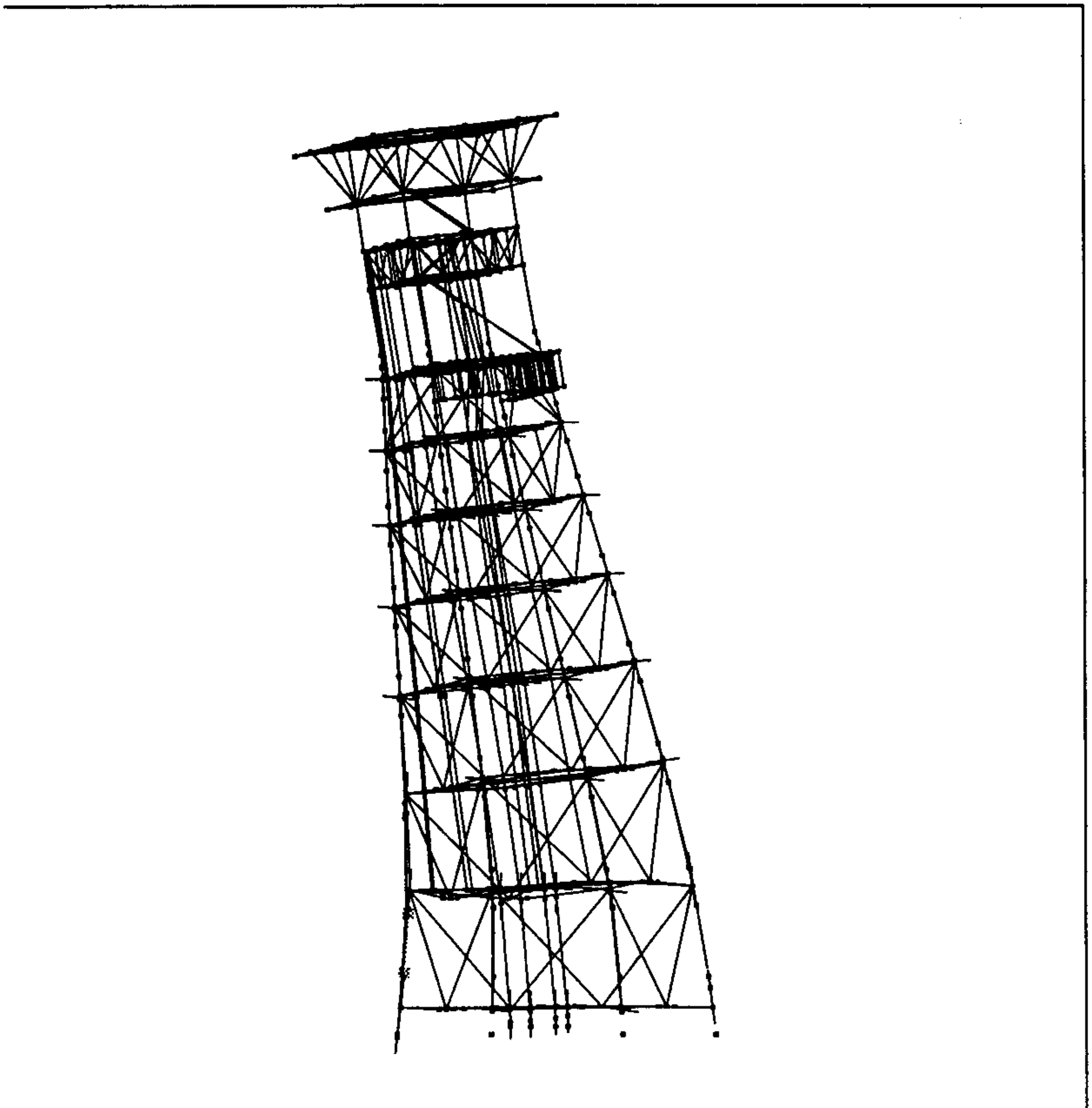
Ultimate Capacity Fixed (Ru–fix) = 3816 kips

Reserve Strength Ratio = 1.6 (fixed base)

Platform Failure Mode = Leg Beam Column



Project: fixbase Model: model1 Version: 0   Tue Aug 23 13:53:13 1994 *



CAP Z X   Figure A.6.2 Fixed Base deflected shape

Inelastic Events Legend

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

CAP Results Table Thu Sep 1 20:55:21 1994

Project: fixbase Model: modell Version: 0

Cut Plane Force Fx - Kip

FIGURE A.6.1 FIXED BASE 315 DEG.

Load Step	X Deck Disp Ft	Force
0	000.000e-3	000.000e-3
1	56.015e-3	72.936
2	0.393	842.192
3	0.731	1610.150
4	1.068	2378.393
5	1.089	2425.721
6	1.098	2446.187
7	1.102	2454.232
8	1.107	2465.163
9	1.112	2474.841
10	1.117	2482.922
11	1.122	2490.860
12	1.126	2498.135
13	1.131	2504.026
14	1.135	2509.079
15	1.140	2516.393
16	1.145	2525.261
17	1.150	2532.915
18	1.154	2540.089
19	1.159	2547.207
20	1.163	2554.118
21	1.168	2561.025
22	1.173	2567.904
23	1.177	2574.784
24	1.182	2581.658
25	1.187	2588.532
26	1.191	2595.404
27	1.196	2602.275
28	1.200	2609.143
29	1.205	2616.011
30	1.210	2622.878
31	1.214	2629.743
32	1.219	2636.608
33	1.223	2643.472
34	1.228	2650.335
35	1.233	2657.197
36	1.237	2664.060
37	1.242	2670.920
38	1.246	2677.781
39	1.251	2684.642
40	1.256	2691.335
41	1.260	2698.277
42	1.265	2705.114
43	1.269	2711.943
44	1.274	2718.772
45	1.279	2725.600
46	1.283	2732.427
47	1.288	2739.255
48	1.292	2746.082
49	1.297	2752.908
50	1.302	2759.734
51	1.306	2766.561
52	1.311	2773.387
53	1.315	2780.211
54	1.320	2787.037
55	1.325	2793.862
56	1.329	2800.688
57	1.334	2807.513
58	1.339	2814.325
59	1.343	2821.146
60	1.348	2827.958
61	1.352	2834.760
62	1.357	2841.555
63	1.362	2848.351
64	1.366	2855.147
65	1.371	2861.940
66	1.375	2868.718
67	1.380	2875.491
68	1.385	2882.258
69	1.389	2889.026
70	1.394	2895.792
71	1.398	2902.559
72	1.403	2909.313
73	1.408	2916.069
74	1.412	2922.825
75	1.417	2929.574
76	1.421	2936.324
77	1.426	2943.067
78	1.431	2949.810
79	1.435	2956.554
80	1.440	2963.295
81	1.444	2970.038
82	1.449	2976.780
83	1.454	2983.522
84	1.458	2990.263
85	1.463	2997.004
86	1.467	3003.745
87	1.472	3010.486
88	1.477	3017.226
89	1.481	3023.967
90	1.486	3030.706
91	1.490	3037.446
92	1.495	3044.185
93	1.500	3050.925
94	1.504	3057.663
95	1.509	3064.402
96	1.513	3071.140
97	1.518	3077.879
98	1.522	3084.589
99	1.527	3091.309
100	1.532	3098.015
101	1.536	3104.721
102	1.541	3111.427
103	1.545	3118.132
104	1.550	3124.830
105	1.555	3131.314
106	1.559	3137.490
107	1.564	3143.574
108	1.568	3149.738
109	1.573	3155.835
110	1.577	3161.913
111	1.582	3167.780
112	1.586	3173.358
113	1.590	3178.770
114	1.595	3184.207
115	1.599	3189.565
116	1.603	3194.916
117	1.608	3200.252
118	1.612	3205.615
119	1.616	3210.888
120	1.621	3216.103
121	1.625	3221.305
122	1.629	3226.442
123	1.634	3231.542
124	1.638	3236.695
125	1.643	3241.784
126	1.647	3246.774
127	1.651	3251.718
128	1.656	3256.661
129	1.660	3261.604
130	1.664	3266.546
131	1.668	3271.490
132	1.673	3276.431
133	1.677	3281.374
134	1.681	3286.315
135	1.686	3291.258
136	1.690	3296.199
137	1.694	3301.141
138	1.699	3306.082
139	1.703	3311.024
140	1.707	3315.963
141	1.712	3320.858
142	1.716	3325.798
143	1.720	3330.710
144	1.725	3335.636
145	1.729	3340.550
146	1.733	3345.462
147	1.738	3350.376
148	1.742	3355.288
149	1.746	3360.201
150	1.751	3365.113
151	1.755	3370.026
152	1.759	3374.937
153	1.764	3379.850
154	1.768	3384.760
155	1.772	3389.672
156	1.777	3394.583
157	1.781	3399.494
158	1.785	3404.405
159	1.790	3409.315
160	1.794	3414.226
161	1.799	3419.137
162	1.803	3424.046
163	1.807	3428.958
164	1.812	3433.885
165	1.816	3438.751
166	1.820	3443.641
167	1.825	3448.417
168	1.829	3453.166
169	1.833	3457.781
170	1.838	3462.445
171	1.842	3467.057
172	1.846	3471.669
173	1.851	3476.283
174	1.855	3480.895
175	1.859	3485.508
176	1.864	3490.120
177	1.868	3494.750
178	1.872	3499.302
179	1.877	3503.908
180	1.881	3508.520
181	1.885	3513.111
182	1.890	3517.699
183	1.894	3522.303
184	1.898	3526.895
185	1.903	3531.487
186	1.907	3536.079
187	1.911	3540.672
188	1.916	3545.261
189	1.920	3549.851
190	1.924	3554.422
191	1.928	3558.573
192	1.933	3562.449
193	1.937	3566.804
194	1.941	3570.352
195	1.946	3573.759
196	1.950	3576.673
197	1.955	3579.340
198	1.960	3583.698
199	1.964	3586.932
200	1.969	3590.465
201	1.973	3591.795
202	1.978	3593.115
203	1.983	3598.724
204	1.988	3603.409
205	1.992	3606.485
206	1.997	3609.855
207	2.002	3612.573
208	2.006	3615.786
209	2.011	3618.555
210	2.016	3622.336
211	2.020	3623.952
212	2.025	3626.560
213	2.030	3631.121
214	2.035	3634.547
215	2.039	3637.334
216	2.044	3640.757
217	2.049	3643.216
218	2.053	3647.020
219	2.058	3648.457
220	2.063	3652.174
221	2.068	3654.847
222	2.072	3658.906
223	2.077	3660.490
224	2.082	3664.066
225	2.087	3667.010
226	2.091	3671.062
227	2.096	3672.753
228	2.101	3676.589
229	2.106	3678.803
230	2.110	3682.485
231	2.115	3684.884
232	2.120	3688.951
233	2.125	3690.000
234	2.129	3694.114
235	2.134	3696.632
236	2.139	3700.425
237	2.143	3702.292
238	2.148	3706.284
239	2.153	3707.587
240	2.158	3711.732
241	2.162	3713.344
242	2.167	3717.838
243	2.172	3718.788
244	2.176	3723.275
245	2.181	3724.673
246	2.186	3728.948
247	2.191	3730.654
248	2.195	3735.273
249	2.200	3736.240
250	2.205	3740.930

251	2.210	3741.793
252	2.215	3746.296
253	2.220	3748.346
254	2.224	3752.953
255	2.229	3754.027
256	2.234	3758.665
257	2.239	3760.282
258	2.243	3765.029
259	2.248	3765.487
260	2.253	3769.946
261	2.258	3772.341
262	2.263	3776.849
263	2.267	3776.813
264	2.272	3781.136
265	2.277	3784.201
266	2.282	3788.256
267	2.286	3788.779
268	2.291	3793.311
269	2.296	3794.474
270	2.301	3798.984
271	2.305	3800.329
272	2.310	3804.878
273	2.315	3805.538
274	2.320	3809.871
275	2.324	3811.967
276	2.329	3816.217
277	2.334	3816.494
278	2.328	3799.252
279	2.324	3789.302
280	2.319	3776.643
281	2.314	3763.813
282	2.230	3559.751
283	2.178	3434.937
284	2.156	3380.923
285	2.146	3357.530
286	2.142	3347.408
287	2.135	3329.868
288	2.132	3321.558
289	2.137	3331.913
290	2.221	3523.016
291	2.247	3580.320
292	2.292	3679.134
293	2.294	3685.281
294	2.299	3692.339
295	2.304	3700.088
296	2.309	3710.219
297	2.314	3721.297
298	2.319	3732.428
299	2.324	3743.016
300	2.329	3752.693
301	2.334	3761.798

*** Table A.6.2 Fixed Base Results Component Failures ***

CAP Inelastic Event Detailed Report Sun Sep 4
00:02:06 1994

Project: fixbase Model: modell Version: 0

Member Name	Load Step	Time	Element Type	Event Description
lga2-60	4	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	4	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	5	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group2-42	40	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lgb1-93	57	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-117	60	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lga2-56	64	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group1-4	66	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lgb1-93	72	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lga2-56	75	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-95	98	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-99	98	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-83	119	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-117	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-99	123	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group1-4	125	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-95	141	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	143	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-107	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-91	165	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group2-79	166	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	166	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-87	166	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-91	167	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-42	169	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-83	178	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	182	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-87	190	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lga2-60	190	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	193	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-87	195	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-87	196	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	197	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-107	197	000.000e-3	Beam Column	Elastic (0,0)
Group3-87	197	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	198	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	198	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-87	198	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-62	200	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group2-58	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-103	201	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	201	000.000e-3	Beam Column	Elastic (0,0)
Group3-91	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-95	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-99	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-91	202	000.000e-3	Beam Column	Elastic (0,0)
Group3-95	202	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	202	000.000e-3	Beam Column	Elastic (0,0)
Group3-91	203	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-95	203	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	203	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	204	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	204	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	207	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-103	208	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	209	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	209	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	210	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	210	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	211	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	211	000.000e-3	Beam Column	Elastic (0,0)
Group3-95	211	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-99	211	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-95	212	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	213	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	213	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	213	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	214	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	215	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	216	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-103	217	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	217	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	218	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	219	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	219	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	219	000.000e-3	Beam Column	Elastic (0,0)
Group3-95	220	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-99	220	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	221	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	221	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-95	221	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	221	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-99	222	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	223	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	223	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	223	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	224	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	225	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	225	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	227	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	228	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lga2-60	228	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group3-107	229	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	230	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	230	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	230	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	231	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	231	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	231	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	232	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	233	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	234	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	235	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	235	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	236	000.000e-3	Beam Column	Elastic (0,0)
Group2-79	238	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-107	238	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	239	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	239	000.000e-3	Beam Column	Elastic (0,0)
Group2-79	240	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	241	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	241	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group2-79	242	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-107	242	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-62	243	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	243	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	244	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-107	244	000.000e-3	Beam Column	Elastic (0,0)
Group2-79	245	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	245	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	246	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-107	246	000.000e-3	Beam Column	Elastic (0,0)
lgb1-97	246	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	247	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	247	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	247	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	248	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-107	248	000.000e-3	Beam Column	Elastic (0,0)
Group2-79	249	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	249	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	250	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	250	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	251	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	251	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	253	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	254	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	255	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	255	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	256	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	257	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	258	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	258	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	259	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	259	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	261	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-58	262	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	262	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group2-79	263	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	263	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	264	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group2-79	265	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	265	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	266	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group2-79	267	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	267	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	268	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	268	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group2-65	269	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group2-79	269	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-97	269	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	270	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	270	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group2-79	271	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lgb1-89	271	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lgb1-97	271	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	272	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
lgb1-97	272	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group2-79	273	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	274	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	275	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	276	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group2-79	277	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
lga2-52	277	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lgb1-89	277	000.000e-3	Beam Column	Elastic (0,0)
r2vd-380	277	000.000e-3	Strut	Strut Buckling
Group1-4	278	000.000e-3	Beam Column	Elastic (0,0)
Group2-42	278	000.000e-3	Beam Column	Elastic (0,0)
Group2-58	278	000.000e-3	Beam Column	Elastic (0,0)
Group2-62	278	000.000e-3	Beam Column	Elastic (0,0)
Group2-65	278	000.000e-3	Beam Column	Elastic (0,0)
Group2-79	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-103	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-117	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-83	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-87	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-91	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-95	278	000.000e-3	Beam Column	Elastic (0,0)
Group3-99	278	000.000e-3	Beam Column	Elastic (0,0)
lga2-52	278	000.000e-3	Beam Column	Elastic (0,0)
lga2-56	279	000.000e-3	Beam Column	Elastic (0,0)
lga2-60	279	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-93	279	000.000e-3	Beam Column	Elastic (0,0)
lgb1-97	279	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lga2-60	280	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
lgb1-97	280	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-60	281	000.000e-3	Beam Column	Elastic (0,0)
lgb1-97	281	000.000e-3	Beam Column	Elastic (0,0)
lga2-60	282	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
Group2-62	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group2-65	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lga2-56	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-60	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
lga2-56	292	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	292	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-60	295	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lga2-52	296	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-60	296	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group3-95	299	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-60	299	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	299	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
Group3-99	300	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lga2-60	300	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group3-103	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-95	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	302	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-87	302	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-91	302	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	302	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group1-4	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-117	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-83	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-87	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lga2-60	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	303	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group2-42	304	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
lga2-60	304	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
lga2-52	306	000.000e-3	Beam Column	Elastic (0,0)
lga2-56	306	000.000e-3	Beam Column	Elastic (0,0)
lgb1-97	306	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-103	307	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	307	000.000e-3	Beam Column	Elastic (0,0)
Group3-91	307	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-99	307	000.000e-3	Beam Column	Elastic (0,0)
lga2-52	307	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
lga2-56	307	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	307	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
Group2-42	308	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group2-79	308	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,1)
Group3-103	308	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-107	308	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	308	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
Group3-91	309	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
Group3-99	309	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
lgb1-97	309	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group2-79	310	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
Group3-107	310	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
Group3-95	310	000.000e-3	Beam Column	Elastic (0,0)
lga2-60	310	000.000e-3	Beam Column	Beam Clmn Fully Plastic (2,2)
Group3-103	311	000.000e-3	Beam Column	Elastic (0,0)
Group3-107	311	000.000e-3	Beam Column	Elastic (0,0)
Group3-83	311	000.000e-3	Beam Column	Elastic (0,0)

Group3-99	311	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	311	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
lgb1-93	311	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	311	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-99	312	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	312	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-93	312	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	312	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-95	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-93	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	313	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-69	314	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-79	314	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-87	314	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	314	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	314	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-93	314	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	314	000.000e-3	Beam Column	Elastic	(0,0)
Group2-58	315	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	315	000.000e-3	Beam Column	Elastic	(0,0)
lga2-56	315	000.000e-3	Beam Column	Elastic	(0,0)
Group2-62	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-79	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-99	316	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-56	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-93	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-97	316	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-79	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-87	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-91	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-95	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	317	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	318	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-103	318	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-107	318	000.000e-3	Beam Column	Elastic	(0,0)
Group3-83	318	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	318	000.000e-3	Beam Column	Elastic	(0,0)
Group3-91	318	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	318	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	318	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-97	318	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group3-103	319	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-93	319	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	319	000.000e-3	Beam Column	Elastic	(0,0)
Group3-103	320	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-52	320	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-93	320	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-97	320	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	321	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group3-83	322	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	322	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group2-58	323	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	323	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	323	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	323	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	323	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-89	323	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-87	324	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-95	324	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	324	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	325	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	326	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-107	326	000.000e-3	Beam Column	Elastic	(0,0)
Group3-83	326	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	326	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	326	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	326	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	326	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-89	326	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-93	327	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	327	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-103	328	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	328	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	329	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
lga2-52	329	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-93	329	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-97	329	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group2-46	330	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-103	330	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	330	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-89	330	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-60	331	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group2-69	332	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	333	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	333	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-99	333	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	334	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	334	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	334	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	334	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-95	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	335	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-97	335	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	336	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-103	336	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-107	336	000.000e-3	Beam Column	Elastic	(0,0)
Group3-83	336	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	336	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	336	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	336	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	336	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	336	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-89	336	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	336	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-103	337	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-93	337	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	337	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgb1-97	338	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group2-79	339	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
lga2-52	339	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-60	339	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-93	339	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-54	340	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-60	340	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-89	340	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	340	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-103	341	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-95	341	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-99	341	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgb1-97	341	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	342	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	342	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group2-79	343	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	343	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	343	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-99	343	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	343	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-97	343	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-107	344	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	344	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-95	344	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	344	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-89	344	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	344	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group3-83	345	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgb1-89	345	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	345	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-83	346	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	346	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-97	346	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group2-79	347	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-103	347	000.000e-3	Beam Column	Elastic	(0,0)
Group3-107	347	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	347	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	347	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	347	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	347	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-89	347	000.000e-3	Beam Column	Elastic	(0,0)
Group3-91	348	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	348	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-93	348	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	349	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-99	349	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	349	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-99	350	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lgb1-89	350	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-93	350	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-103	351	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	351	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-95	351	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-60	351	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-79	352	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-83	352	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	352	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	352	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	352	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	352	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-89	352	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-93	352	000.000e-3	Beam Column	Elastic	(0,0)
Group3-103	353	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-56	353	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-79	354	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-103	354	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-52	354	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-56	354	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	355	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
lgb1-89	355	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-93	355	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	356	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-95	356	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	356	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	356	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lgb1-97	356	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-91	357	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lgb1-97	357	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-107	358	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	358	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	358	000.000e-3	Beam Column	Elastic	(0,0)
Group3-107	359	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	359	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	359	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group2-79	360	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-107	360	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	360	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	360	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	360	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-99	361	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-79	362	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-103	362	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-83	362	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	362	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	362	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	362	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-79	363	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	363	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-87	363	000.000e-3	Beam Column	Elastic	(0,0)
Group3-95	363	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	363	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	363	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	364	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-83	364	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	364	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	364	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-99	364	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	364	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-103	367	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-60	367	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-103	368	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	368	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group3-107	372	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-54	373	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	373	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-65	374	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	374	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-107	375	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	376	000.000e-3	Beam Column	Elastic	(0,0)
Group3-107	377	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	377	000.000e-3	Beam Column	Elastic	(0,0)
Group3-107	378	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	378	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-107	379	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-87	379	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	380	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-107	380	000.000e-3	Beam Column	Elastic	(0,0)
Group3-91	381	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	382	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group3-107	382	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	382	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-54	383	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-65	383	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-79	383	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-103	383	000.000e-3	Beam Column	Elastic	(0,0)
Group3-107	383	000.000e-3	Beam Column	Elastic	(0,0)
Group3-83	383	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	383	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	383	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)

Group3-95	383	000.000e-3	Beam Column	Elastic	(0,0)
Group3-99	383	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	383	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-89	383	000.000e-3	Beam Column	Elastic	(0,0)
Group2-42	384	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-46	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-54	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-58	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-62	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-65	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-69	384	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	384	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	384	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-56	384	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	384	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
lgb1-93	384	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	384	000.000e-3	Beam Column	Elastic	(0,0)
r2vd-380	384	000.000e-3	Strut	Strut Reloading	
Group2-42	385	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group2-69	385	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	385	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	385	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
r2vd-380	385	000.000e-3	Strut	Strut Buckling	
Group2-46	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-54	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-58	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-62	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-65	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-79	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-56	386	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	386	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
lga2-60	388	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,2)
Group3-91	389	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-60	389	000.000e-3	Beam Column	Beam Clmn Fully Plastic	(2,2)
Group2-50	390	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-66	390	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
ravd-223	390	000.000e-3	Strut	Strut Buckling	
Group2-46	391	000.000e-3	Beam Column	Elastic	(0,0)
Group2-50	391	000.000e-3	Beam Column	Elastic	(0,0)
Group2-54	391	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-69	391	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group3-87	391	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-91	391	000.000e-3	Beam Column	Elastic	(0,0)
lga2-56	391	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lgb1-97	391	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
ravd-223	391	000.000e-3	Strut	Strut Reloading	
Group2-46	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-50	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-54	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group2-69	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-56	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	392	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
Group1-4	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-42	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-46	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-50	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-54	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-58	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-62	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-65	393	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-69	393	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	393	000.000e-3	Beam Column	Elastic	(0,0)
Group3-117	393	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	393	000.000e-3	Beam Column	Elastic	(0,0)
lga2-56	393	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	393	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-66	393	000.000e-3	Beam Column	Elastic	(0,0)
lgb1-97	393	000.000e-3	Beam Column	Elastic	(0,0)
r2vd-380	393	000.000e-3	Strut	Strut Reloading	
Group1-4	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group2-42	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-62	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
Group2-69	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-79	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-117	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group3-87	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
Group3-91	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
lga2-52	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(0,1)
lga2-56	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)
lga2-60	394	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,1)
ravd-223	394	000.000e-3	Strut	Strut Buckling	
Group1-4	395	000.000e-3	Beam Column	Elastic	(0,0)
Group2-42	395	000.000e-3	Beam Column	Elastic	(0,0)
Group2-62	395	000.000e-3	Beam Column	Elastic	(0,0)
Group2-65	395	000.000e-3	Beam Column	Elastic	(0,0)
Group2-69	395	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	395	000.000e-3	Beam Column	Elastic	(0,0)
Group3-117	395	000.000e-3	Beam Column	Elastic	(0,0)
Group3-87	395	000.000e-3	Beam Column	Elastic	(0,0)
Group3-91	395	000.000e-3	Beam Column	Elastic	(0,0)
lga2-52	395	000.000e-3	Beam Column	Elastic	(0,0)
lga2-56	395	000.000e-3	Beam Column	Elastic	(0,0)
ravd-223	395	000.000e-3	Strut	Strut Reloading	
Group1-4	396	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
Group2-42	396	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group2-79	396	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,0)
Group3-117	396	000.000e-3	Beam Column	Beam Clmn Initial Yield	(2,0)
ravd-223	396	000.000e-3	Strut	Strut Buckling	
Group1-4	397	000.000e-3	Beam Column	Elastic	(0,0)
Group2-42	397	000.000e-3	Beam Column	Elastic	(0,0)
Group2-79	397	000.000e-3	Beam Column	Elastic	(0,0)
Group3-117	397	000.000e-3	Beam Column	Elastic	(0,0)
lga2-60	397	000.000e-3	Beam Column	Beam Clmn Initial Yield	(1,1)

PARTICIPANT "K" RESUBMITTAL

1.0 SUMMARY

This document presents the results of the ultimate strength analyses for the benchmark jacket, undertaken using SAFJAC. The analyses are based on the Benchmark Basis Document from PMB dated 24 February 1994 and the four subsequent revisions.

SAFJAC, for the Strength Analysis of Frames and JACkets, is a specially developed nonlinear analysis program for calculating the reserve and residual strength of framed structures consisting of tubulars, I-beams, box and rectangular sections. The program can predict large displacement behaviour of two and three dimensional frames including the effect of buckling, material plasticity and joint nonlinearity.

SAFJAC was developed as part of the Frames Project JIP with sponsorship from nine operators and the HSE. It has been extensively verified against test data and standard simple solutions and elastic progressive collapse (member replacement) analyses of complex structures. It is now used in practice to perform progressive collapse analyses for structures under extreme environmental loads or subject to accidental damage or overload.

The SAFJAC computer model was converted directly from a SACS/SEASTATE model to generate structural dead and environmental loads. The model contains all primary jacket members, piles, conductors, risers, boat landing and bumpers, with a simplified deck structure including four horizontal plans and four deck legs. Each member was modelled using a quartic elastic beam-column element to include large deflection behaviour with automatic subdivision either into cubic elastic-plastic elements or plastic hinge elements on detection of plasticity.

Element types used in the benchmark analyses are:

- Quartic elastic beam-column element modelling geometric nonlinearity with automatic subdivision into cubic element on detection of plasticity.
- Cubic elastic-plastic beam column with Gauss integration and monitoring points over the sections at the two Gauss points.
- Quartic elastic beam-column element with automatic subdivision into plastic hinge elements.
- Piecewise linear joint element with decoupled translational and rotational nonlinear springs.

Benchmark ultimate strength analyses were performed for three wave approach directions, those directions being defined by waves on headings of 180°, 225° and 270° with respect to platform North (measured clockwise). A series of analyses was carried out and six results are presented in detail in this report.

For the 'required' benchmark case, the piles below mudline were modelled using beam-column elements and the axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using piece-wise linear spring elements. API static soil criteria were used. The jacket was initially modelled with no account of joint flexibility. Results are presented for the 225° and 270° wave approach directions. The resistance of the foundations to the conductors was also modelled (with no gap

allowance) and this led to local failures in the plan bracing at the mudline (Elevation -157 ft.) due in part to the simplified modelling of the conductor framing. Results are presented in Table 1, below.

The voluntary analyses were undertaken for the jacket model rigidly fixed at Elevation -169 ft. This gave the opportunity to explore the dependence of ultimate strength predictions on various modelling assumptions which would otherwise be masked by the dominance of the foundation driven failure for this installation. Four additional analyses are reported here to enable the following comparisons with respect to ultimate capacity and RSR to be made. The results are reported in the table and the figures in brackets indicate the analyses forming the basis of the comparison.

- Variation for different wave approach directions (RJRF - 180°, 225°, 270°)
- Relative influence of the weak foundation (RJRF and RJPF - 225°, 270°)
- Relative influence of joint flexibility and limiting capacity (RJRF and FJRF - 270°)
- Influence of nonlinear element modelling (RJRF - 270° - see Section A.5)

Analysis	Required		Voluntary			
	225° wave	270° wave	180° wave	225° wave	270° wave	
Load level	RJPF *	RJPF	RJRF V3	RJRF V2	RJRF V1	FJRF V4
Ultimate capacity (Kips)	1689	2039	4196	4740	4046	4077
Section 17 Loads - Ls17u (Kips)	2830	2702	1880	2830	2702	2702
Ref. Loads API 20th (Kips)	2420	2268	1590	2420	2268	2268
RSR	0.70	0.90	2.64	1.96	1.78	1.80

Note:
RJPF - Rigid joints and piled foundation
RJPF * - Results submitted on 23 September ie. after initial submission but prior to JIP draft report issue
RJRF - Rigid joint and rigid foundation
FJRF - Flexible joints and rigid foundation

It is recognised that maximum base shears occur for the 245° wave direction (Ls17u = 2928 kips and API RP 2A 20th Edition load = 2468 kips). A response and reserve strength similar to the 225° result would be anticipated.

Table 1: Summary of ultimate strength and RSR for Benchmark platform

(Note: directions here and throughout report are referenced with respect to platform North).

PART A: BENCHMARK ANALYSIS

A.1 Environmental Criteria

Environmental criteria selected are based on the Draft Guidelines (API RP-2A WSD) Section 17. The basis of the environmental loads for each of the three approach directions is provided in Table 2. Arising from discussions at the JIP meeting on 19 November, it is now recognised that the directional wave/current factors had been interpolated in error. This fact is merely noted here and the analyses reported in this document relate to the parameters given in Table 2.

Environmental data	Approach directions		
Orientation wrt Platform North	180°	225°	270°
Wave height (ft)	55.83	66.84	66.91
Wave period (sec)	13.5	13.5	13.5
Current profile (ft/sec)	3.76	3.87	3.82
Storm surge (ft)	3.0	3.0	3.0
Wind speed @ 10m above MSL (ft/sec)	143.3	143.3	143.3
Wave load in deck (kips)	39.9	197.7	249.6
Wave crest elevation above mudline (ft)	194.1	202.6	202.6

Table 2: **Environmental data for the benchmark analyses**

Wave and current deck forces were determined based on the draft guideline (API RP-2A WSD) Section 17 and the projected areas provided by PMB. Wave crest and current velocities were applied on the projected areas at and below the wave crest based on spreadsheet calculations. A resultant force was calculated for each deck area and applied at the centre. That force was applied to nodal joints at the incident face. If the deck was completely engulfed the force was distributed equally to joints above and below, otherwise upper joints received proportionately less.

A.2 3-D Model Generation

The benchmark jacket was modelled using quartic elastic-plastic beam-column elements. Taking advantage of the techniques of automatic mesh refinement employed within SAFJAC and the high order quartic beam element formulation to model beam-column behaviour accurately, each member was modelled using a single quartic element. Extra nodes and elements were introduced on the legs to model leg cans. The model contains all primary and major secondary jacket members, piles conductors, risers, boat landing and bumpers and a simplified deck including four horizontal deck plans and four deck legs.

The piles inside the legs were modelled using quartic elements. Elements were introduced between the piles and legs at each of the horizontal plan levels to restrain the lateral movement of the pile to that of the leg. The gaps between legs and pile shims were ignored.

Well conductors were modelled using quartic elements. Elements were introduced between the conductors and conductor guides to restrain the lateral movement of the conductors to that of the jacket. The gaps between conductors and conductor guides were ignored given the uncertainty of gap size and initial conductor position.

For the required ultimate strength analysis, piles and conductors below mudline were modelled using quartic elements with subdivision into plastic hinge elements on detection of plasticity to approximate large displacement plastic behaviour of the piles or conductors as the wave loading is increased in the analysis. The axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using piece-wise linear spring elements to account for the soil-pile interactions. The soil P-Y, T-Z and Q-Z data generation was based on the data supplied by PMB and the guidelines in API RP-2A 20th Edition which were implemented using SACS. In the voluntary analyses piles were assumed to be fixed at Elevation -169 ft and conductors were assumed to be supported vertically at the mudline.

Initial analyses were performed with all joints rigidly connected. The code checking facilities of SAFJAC (to API RP 2A lower bound with safety factors removed), highlighted early joint 'failures' and to demonstrate the influence of joint flexibility and limiting strength on the system response, K joints were modelled in the last voluntary analysis using nonlinear springs. The joint character-istics were determined using a SAFJAC database of nonlinear joint flexibilities in conjunction with the database and mean capacity equations underlying the HSE Guidance Notes equations published in Offshore Technology Report, OTH 89 308.

The material yield strength (mean) has been taken as 42 ksi for all jacket, deck and foundation members. The Section 17.0 ultimate starnegth loads were applied at nodal points throughout the structure enabling proper account to be taken of incident loads on risers, conductors etc.

A.3 Software Description

SAFJAC was specifically developed for the nonlinear Strength Analysis of Frames and JACkets. The program was developed and enhanced through BOMEL's Frames Project and is now used in practice for the assessment of existing structures and in specialist design studies. The program can predict large displacement behaviour of two and three dimensional frames, including the effects of buckling, material plasticity and joint nonlinearity. Its development focused on structural idealisation and efficient nonlinear solutions that do not over-simplify member or joint behaviour and that do not depend on assumptions by the user, for example, of where yielding occurs. Particular features of SAFJAC are itemised below with comments on the use of features in the benchmark analysis following:

- Full member cross-section library including tubulars, I-sections and rectangular hollow sections.
- High order elastic beam-column element requiring only one element over the full member length to model buckling accurately, allowing for the end fixity provided by the structural continuity (ie. no requirement for specification of effective lengths).

- Continuous monitoring for onset of plasticity
- Automatic subdivision of members where plasticity occurs without interruption of solution. Automatic insertion of plastic hinges or distributed plasticity elements on member subdivision.
- Alternative approaches to plasticity depending on sensitivity of solution and need for close monitoring of stiffness changes.
- Ability to control overall structural displacement rather than load in order to follow the post ultimate unloading behaviour.
- Separate definition of joint flexibility and nonlinearity by nonlinear joint springs (up to six degrees of freedom per node).
- Availability of extensive database of nonlinear joint properties.
- Full nonlinear foundation modelling.

Element types
A comprehensive set of elements is available and these are summarised in Table 3.

Element	Description
Type 31	Cubic elastic-plastic beam-column element with Gauss integration and monitoring points over the sections at the two Gauss points.
Type 32	Quartic elastic beam-column element with automatic subdivision upon detection of plasticity. On subdivision, Type 31 elements are inserted in the plastic zones.
Type 33	Quartic elastic beam-column element. For large displacement response of a complete member.
Type 34	Quartic plastic hinge beam-column element with automatic subdivision into two elements if a plastic hinge is detected within the element length.
Type 41	Piece-wise linear joint element with decoupled translational and rotational nonlinear springs.

Table 3: Element types within SAFJAC

Pile-soil interaction
Full pile soil interaction was modelled using beam-column elements for the piles and nonlinear springs to represent the P-Y, T-Z and end bearing soil stiffness. Similar modelling techniques were used for the conductors.

Solution procedure
In the ultimate strength analysis, loading is applied in two stages. Initial static loading representing the still water condition is applied prior to the application of the proportional load due to environmental conditions, including current, wave and wind loads. This allows the still

water condition of the jacket to be initialised prior to the application of any environmental loads. The proportional loads are then applied incrementally until the maximum proportional load factor is reached. The post peak response is obtained by pushing the jacket further until the user-prescribed displacement on a certain node is reached. To overcome convergence problems in the vicinity of the ultimate load, both load and displacement control of the solution were used.

API joint checks
Tubular joint checks to the 20th Edition of API RP2A can be carried out at any stage of the nonlinear analysis. The joints are automatically classified according to their geometry and the load path. Safety factors are removed and a utilisation of unity indicates that the joint load has reached the API lower bound capacity.

Joint behaviour
Nonlinear joint behaviour and strength can be modelled using piecewise linear springs with differing behaviour in tension and compression. Separate springs may be defined for axial, in-plane and out-of-plane bending behaviour.

A.4 Ultimate Strength Analysis (required)

The full platform model including foundation effects is submitted as the required case. Results are presented for the wave approach directions of 225° and 270° with respect to platform North. These are given on the following pages in the form of a load-displacement plot and tabulated data as requested. The 270° results were submitted on 22 August 1994 and the 225° results followed on 23 September (ie. after the initial deadline but prior to the issue of the PMB draft JIP report).

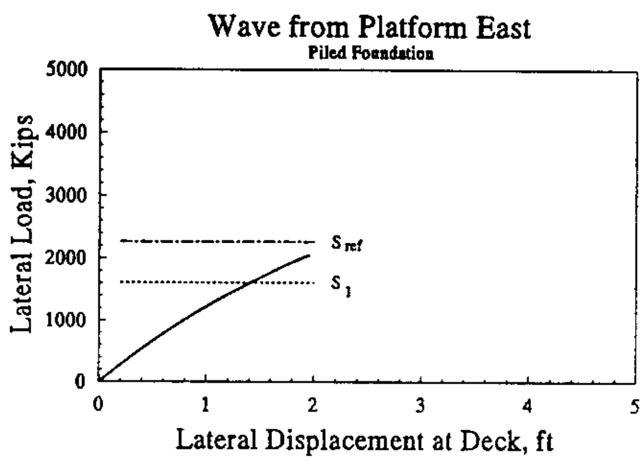
Analysis R1
Figure 1 gives the overall load deflection response and the required capacity information. Table 2 gives datapoints from which the global response is plotted and the first member failure is indicated. Further failures are more conveniently indicated in Figure 2 which gives the order of hinge formation and the load factor (with respect to the reference level). These sites of plasticity can be linked to the local deformation at the peak load shown in Figure 3.

In the platform model, the piles below mudline were included using beam-column elements and the axial response (T-Z curves) and the lateral response (P-Y curves) of the soil were modelled using spring elements. The jacket was initially modelled with no account of joint flexibility. The results are presented for the analysis using element Type 34 which introduces a plastic hinge (with no strain hardening) on detection of plasticity.

A number of modelling variations were also implemented to test the robustness of the solution. An alternative analysis incorporated element Type 31 where automatic subdivision introduces a cubic element (instead of the plastic hinge approximation) to model the gradual spread of plasticity though the thickness and along the member length taking account of strain hardening. A third analysis included the leg extensions below the mudline which had previously been omitted and a fourth adopted cyclic, in place of static, P-Y clay characteristics. In all cases the limit load and general mode of failure remained the same.

The soft foundation allowed large axial and lateral movements of the piles contributing to large deflections at the top of the jacket. The rigid body rotation is a significant component as seen in Figure 3. The resistance of the foundations to the conductors was also modelled (with no gap allowance) and this led to local failures in the plan bracing at the mudline (Elevation -157 ft.). The first hinge forms at a load factor of 0.41 corresponding to a lateral load of 930Kips. Load factors shown relate to the reference level load, although the Section 17.0 loads were applied and factored in the analaysis. A further four hinges occurred by the load factor of 0.70 and at 0.71 the first member has double hinges (hinges 3 and 6) and therefore fails in compression as shown in Figure 2. The maximum load factor (RSR) predicted is only 0.899, at which point a total of 16 hinges have developed in the plan at Elevation -157ft.

Figures 2 and 3 demonstrate the extent of failure in the plan and tubular joint code checks indicate that the connections would also fail. The load factors corresponding to the API RP 2A lower bound are shown in Figure 2. The sites of overstress in members and joints are similar, indicating the inadequacy of the bracing system as modelled to sustain the loads/deformations associated with the conductor attachments. Additional non-compliances can be found in the plan connections in the bay above at the end of the analysis. These code checks have been based on the member properties because of the lack of clarity in the drawings. Any can details which would strengthen the joints have therefore been ignored. Even without explicit account of the tubular joints, the platform fails to sustain the assessment load required by Section 17.0 to demonstrate fitness-for-purpose. This is driven by the soft foundation and for that reason detailed modelling of the joint characteristics is not meaningful. In subsequent analyses properties of the conductor framing were more accurately represented. Whilst this influenced the sites of overstress in the jacket structure the global response was still dominated by the weak foundation.



Ls17u - Sec. 17.0 Ultimate Strength Load	2702 Kips
Reference Level Load	2268 Kips
Load Level at First Member Failure	1605 Kips
Ultimate Capacity (Ru)	2043 Kips
Reserve Strength Ratio	0.901
Platform Failure Mode	Foundation T-Z

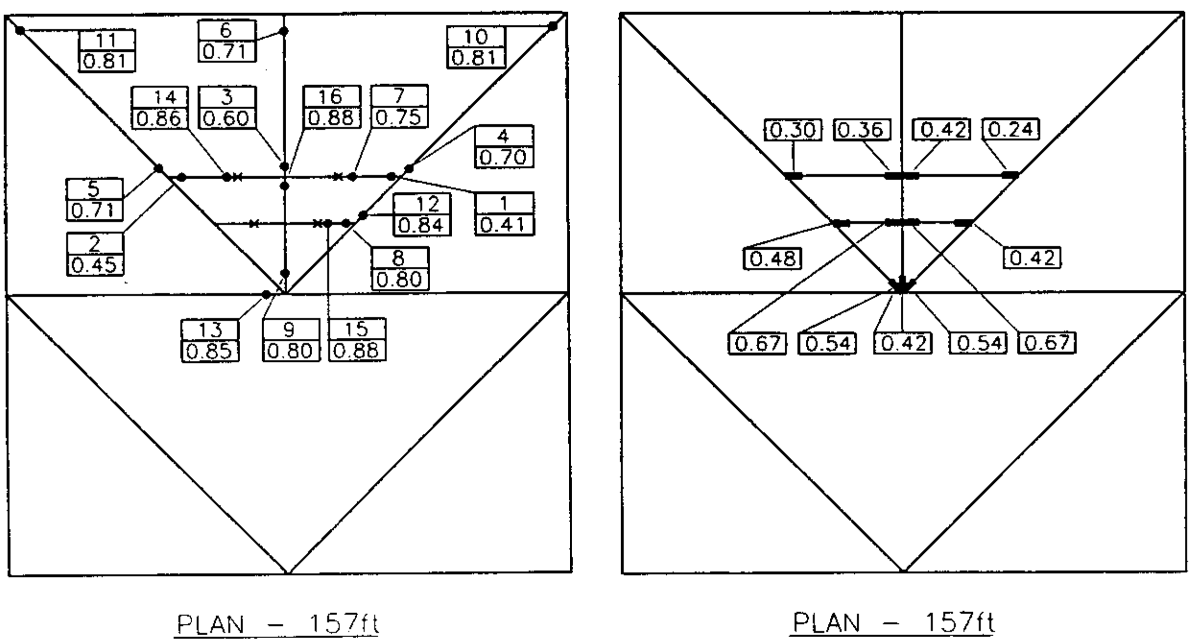
Figure 1: Ultimate Strength Analysis - Minimum Required Results
Load displacement and failure mode results
for wave approach direction at 270° wrt Platform North

Lateral load level at failure of the first member by yielding in the mudline plan bracing 1605 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.416	553		
3	0.791	996		
4	1.027	1245		
5	1.410	1605	1	Yield (Hinges 3 and 6)
6	1.631	1799		
7	1.802	1938		
8	1.949	2039		

* See Figure 2 indicating location and sequence of component failures

Table 4: Ultimate Strength Analysis - Minimum Required Results
Load displacement and failure mode results
for wave approach direction at 270° wrt Platform North



× CONDUCTOR LOCATION

● PLASTIC HINGE FORM

0.36 — LOAD FACTOR AT WHICH JOINT IR > 1.0

1 — HINGE No.

0.41 — LOAD FACTOR WITH RESPECT TO REFERENCE LEVEL LOAD

Figure 2: Ultimate Strength Analysis - Minimum Required Results
Location and sequence of component failures for wave approach
direction at 270° wrt Platform North

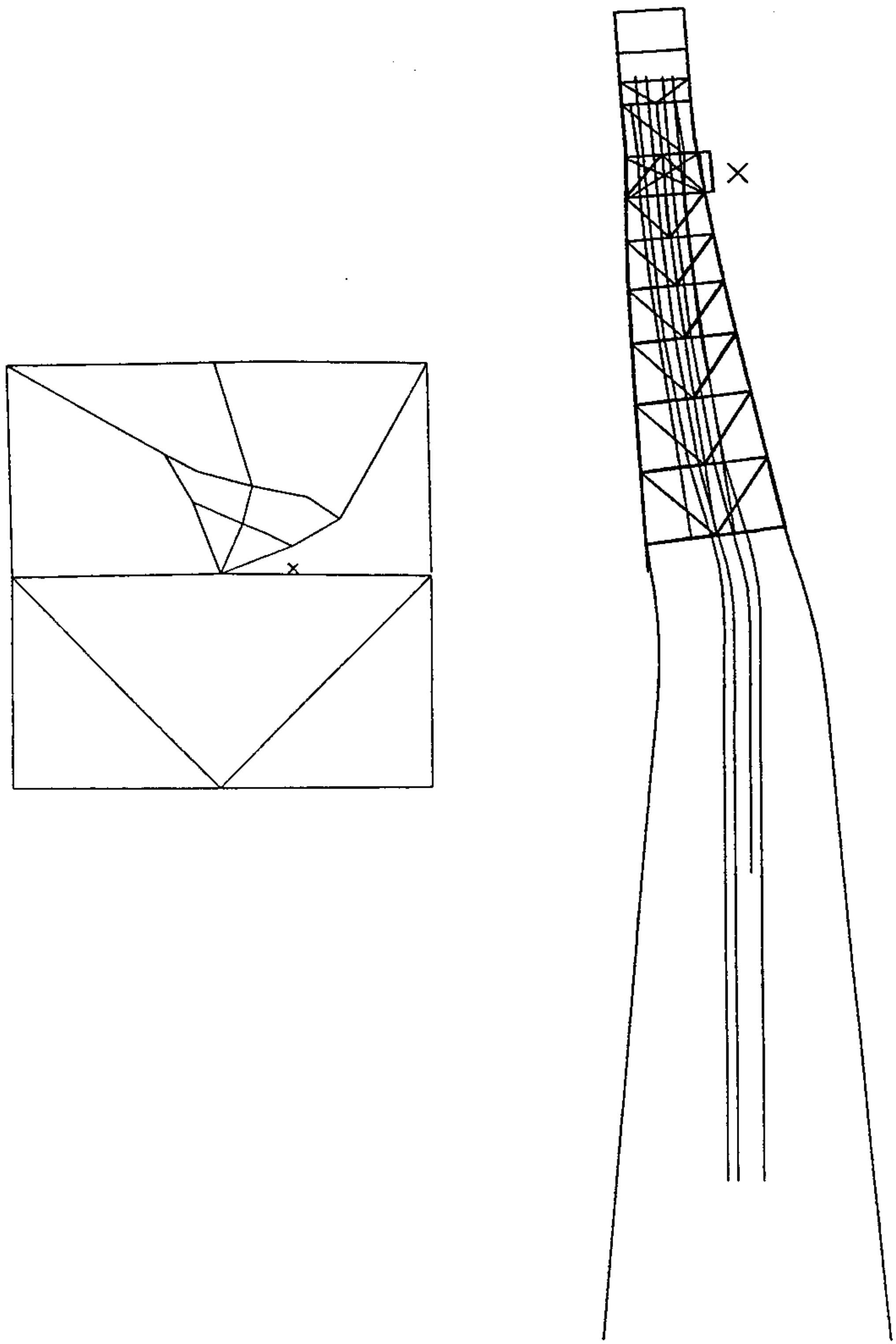


Figure 3: Ultimate Strength Analysis - Minimum Required Results
Deflected shape of jacket and -157 ft plan bracing on attainment of peak capacity for wave approach direction at 270° wrt Platform North (mag. x20)

Analysis R2
The platform was reanalysed for the wave approach direction of 225° with respect to platform North. It is noted that for the 245° wave approach direction, the Section 17.0 ultimate strength load is 3.5% higher than for the 225° direction. This case has not been reanalysed but the dominance of the foundation failure and the level of 'reserve' strength is expected to be similar.

Figure 4 shows the overall load deflection response and required capacity information for the 225° case. Table 5 gives data points from which the global response is plotted and the first member failure is indicated. The order and sequence of plastic hinge formation with indication of the load factors (with respect to the reference load level) are given in Figure 5.

As for Direction 3, the soft foundation allowed large axial movements of the compression and tension piles and caused large deflections at the top of the jacket. The rigid body rotation is a significant component as seen in Figure 6. Table 6 gives the forces in each of the T-Z springs for the peak load factor, compared to the residual capacity of pile skin frictions. Spring numbered 27 represents the end bearing. For the compression pile 1, springs 1 to 22 are fully utilised. Springs 23 to 26 are either close to T_{max} or have started to unload. Based on very simple hand calculations of overturning capacity using T-Z spring forces, the foundation has reached 93% of its capacity.

In this model the T-Z residual resistance for clays was based on the API RP 2A lower bound value $T_{res}=0.7T_{max}$. Subsequent analyses undertaken after the PMB draft report was received are contained in the Annex and demonstrate that adopting the upper plateau ($T_{res}=0.9T_{max}$) imparts some 27% additional capacity to the platform.

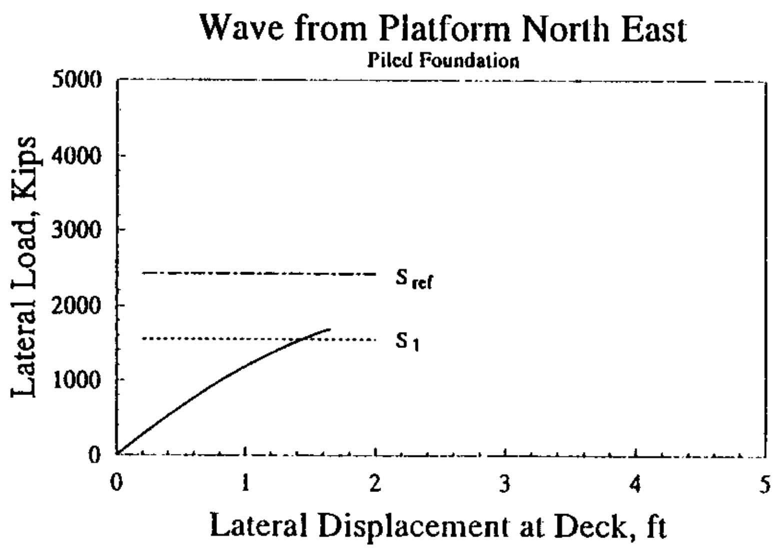
The lateral load level at failure of the first member by buckling in the mudline plan bracing is 1549 kips.

Figure 5 shows that the first hinge forms at a load factor of 0.35 corresponding to a lateral load of 847 kips. By a load factor of 0.64 (ie. a lateral load of 1549 kips) the first member fails in combined compression and bending, as indicated by hinges 3, 4 and 6 shown in Figure 5. The reserve strength ratio predicted is 0.70, at which point a total number of 8 hinges have developed in the plan at Elevation -157ft.

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures *	Structural Component Failure Mode
1	0.000	0		
2	0.154	210		
3	0.379	502		
4	0.821	1005		
5	1.201	1359		
6	1.436	1549	1	Comp/bend (Hinges 3, 4 & 6)
7	1.493	1590		
8	1.506	1600		
9	1.587	1655		
10	1.650	1689		

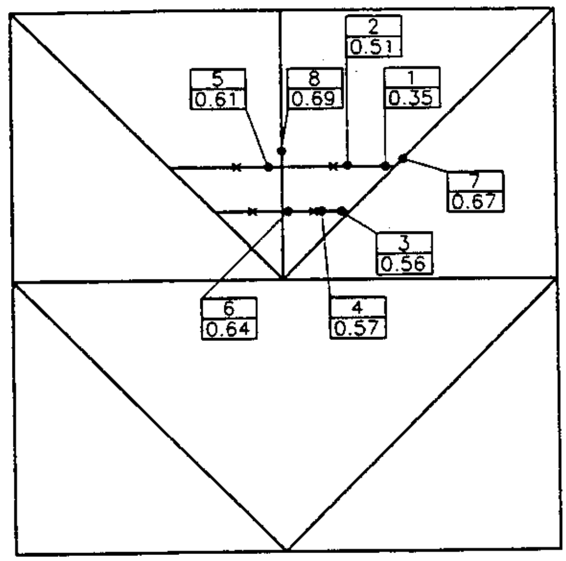
* See Figure 5 indicating location and sequence of component failures

Table 5: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for piled foundation case with no joint flexibility (V5) for wave approach direction at 225° wrt N



Ls17u - Sec. 17.0 Ultimate Strength Load	2830 Kips
Reference Level Load	2420 Kips
Load Level at First Member Failure	1549 Kips
Ultimate Capacity (Ru)	1689 Kips
Reserve Strength Ratio	0.698
Platform Failure Mode	Foundation T-Z

Figure 4: Ultimate Strength Analysis - Required Results
Load displacement and failure mode results for piled foundation case with no joint flexibility (R2) for wave approach direction at 225° wrt N



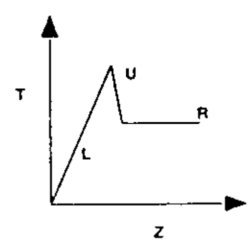
× CONDUCTOR LOCATION

• PLASTIC HINGE FORM

1 — HINGE No.
0.42 — LOAD FACTOR WITH RESPECT TO REFERENCE LEVEL LOAD

Figure 5: Ultimate Strength Analysis - Required Results
Location and sequence of component failure for piled foundation case
with no joint flexibility (R2) for wave approach direction at 225° wrt N

Spring	T_residual (Kips)	Pile - 1 (Kips)	Pile - 2 (Kips)	Pile - 3 (Kips)	Pile - 4 (Kips)
1	13.08	-13.36 R	-18.37 L	-18.35 L	13.36 R
2	13.08	-13.36 R	-17.97 L	-17.94 L	13.36 R
3	13.08	-13.36 R	-17.58 L	-17.55 L	13.36 R
4	13.08	-13.36 R	-17.04 L	-16.95 L	13.36 R
5	28.10	-28.11 R	-33.92 L	-33.74 L	28.11 R
6	28.10	-28.11 R	-32.54 L	-32.37 L	28.11 R
7	28.10	-28.11 R	-31.27 L	-31.10 L	28.11 R
8	28.10	-28.11 R	-30.07 L	-29.91 L	28.11 R
9	42.19	-42.24 R	-43.27 L	-43.05 L	42.24 R
10	42.19	-42.24 R	-41.16 L	-40.94 L	42.24 R
11	42.19	-42.24 R	-38.78 L	-38.59 L	42.24 R
12	42.19	-42.24 R	-36.25 L	-36.08 L	42.24 R
13	56.07	-56.05 R	-45.09 L	-44.88 L	56.05 R
14	56.07	-56.05 R	-41.88 L	-41.69 L	59.70 U
15	56.07	-56.05 R	-37.83 L	-37.50 L	64.22 U
16	56.07	-56.05 R	-32.41 L	-32.13 L	68.55 U
17	71.11	-71.10 R	-34.84 L	-34.54 L	92.20 U
18	71.11	-71.10 R	-29.32 L	-29.06 L	97.12 U
19	71.11	-71.10 R	-24.38 L	-24.17 L	101.44 L
20	71.11	-71.10 R	-19.93 L	-19.76 L	94.44 L
21	224.00	-224.00 R	-71.63 L	-71.01 L	224.00 R
22	224.00	-224.00 R	-53.90 L	-53.44 L	224.00 R
23	299.45	-323.51 U	-32.35 L	-32.07 L	244.57 L
24	299.45	-385.60 U	-19.96 L	-19.79 L	172.62 L
25	299.45	-426.83 L	-13.13 L	-13.01 L	115.42 L
26	299.45	-391.29 L	-9.94 L	-9.86 L	90.31 L
27(E.B.)	89.07	-34.80 L	-1.52	-1.51 L	0.00
Total	2577.07	-2853.47	-826.33	-820.99	2039.48



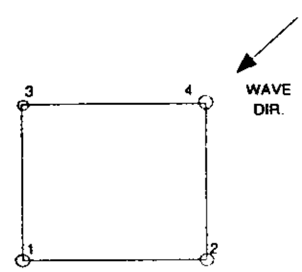


Table 6: Ultimate Strength Analysis - Required Results
Foundation T-Z capacity for piled foundation case
with no joint flexibility (R2) for wave approach direction at 225° wrt N

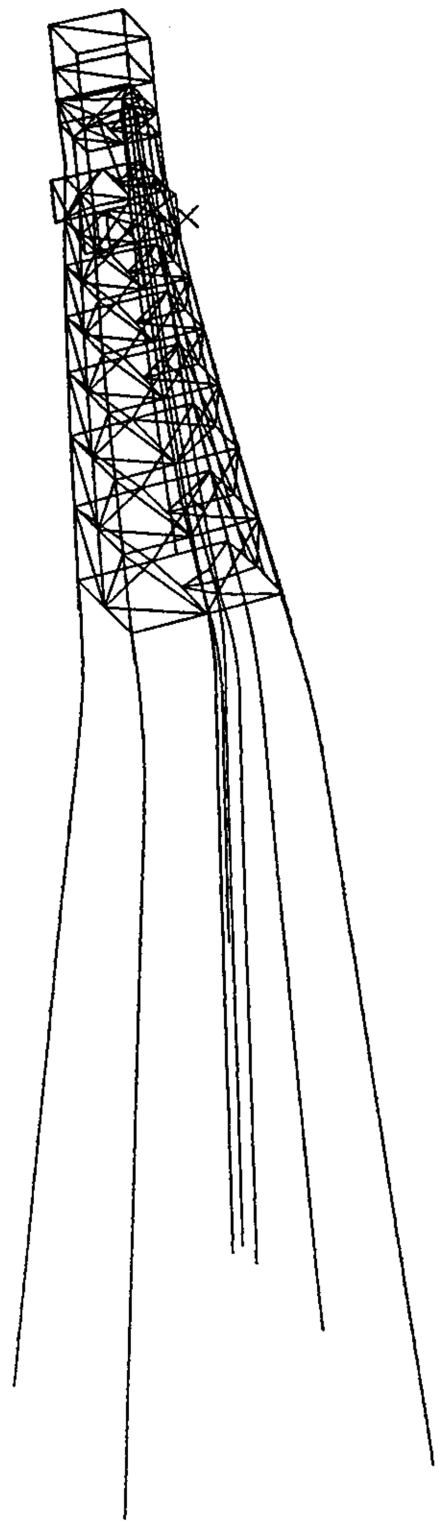


Figure 6: Ultimate Strength Analysis - Required Results
Deflected shape of jacket at peak load for rigid foundation case with no joint flexibility (R2) for wave approach direction at 225° wrt N (mag x 20)

A.5 Ultimate Strength Analysis (Voluntary)

Additional analyses have been performed for the benchmark jacket. In all cases the foundation characteristics were omitted by fixing the jacket at -169 feet. In this way failures associated with the weak foundation were suppressed to enable the ultimate response characteristics of the jacket to be investigated which provides a valuable comparison for benchmark purposes. The other principal differences from the 'Required' case are summarised in Table 7.

Analysis	Difference from 'Required'	Objective
V1	Piles fixed at -169ft. conductors vertically supported at mudline. Quartic elements - subdivision into cubic. Wave approach 270° as 'required' case.	Effect of fixed base/foundation compliance on platform ultimate capacity.
V2	As V1 but wave at 225°	Reserve strength comparisons for alternative wave directions.
V3	As V1 but wave at 180°	
V4	As V1 but K joints modelled using nonlinear springs	Effect of joint behaviour on ultimate capacity.

Table 7: Comparison of voluntary analysis with required case

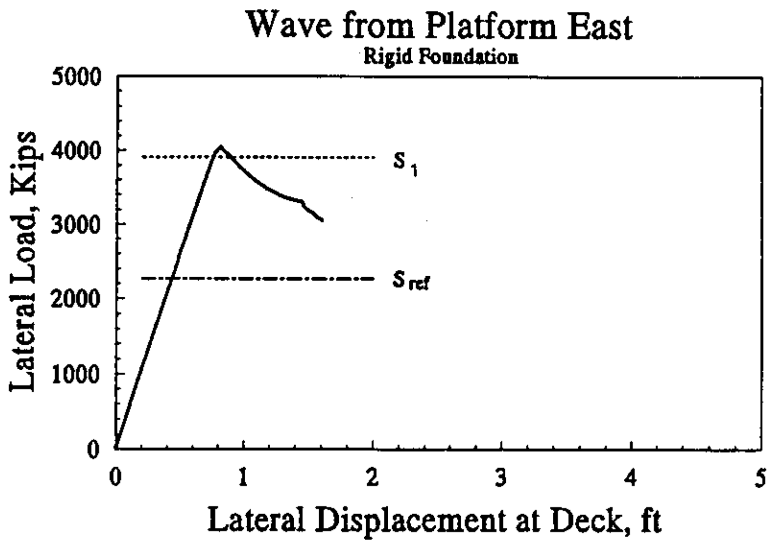
Analysis V1

Figures 7 to 9 and Table 8 present the results for case V1. Details of component failures are only itemised up to the peak load for clarity.

In this first analysis the joint flexibility is ignored but tubular joint checks to API RP 2A 20th Edition are implemented to check whether the lower bound capacity has been exceeded. Member failures are concentrated in the K bracing in the face frames parallel to the incident wave direction. The response is asymmetric for a number of reasons: the current has a component perpendicular to the wave direction: the conductors are offset to one side of the jacket. As a result initial failures occur in the platform North face. Initial failure occurs at a load level of 3910 Kips (Load factor with respect to reference load of 1.72). Thereafter a series of member failures occur until the ultimate jacket strength is attained at a reserve strength ratio of 1.78.

The Type 31 distributed plasticity elements spawned from the initial Type 32 selection successfully model the sequence of failures and the post-buckling curve. An alternative plastic hinge analysis was also performed using Type 34 elements (Table 3). Figure 10 compares the solutions from the two analyses demonstrating the close agreement in the peak load predictions but the post-failure differences arising from the absence of material strain hardening in the idealised plastic hinge approach and the gradual development of plasticity in the cubic elements.

The joint code checks at a load factor of 1.0 (ie. at the reference level load) determine that the loads through the K joints at levels -126ft and -97ft on both face frames and at -71ft on the South frame. exceed the lower bound capacity implicit in API RP 2A provisions. Local increased can wall thicknesses are included in the code checks. The first noncompliance therefore occurs before the initial member failures. The K joints at upper levels become stressed beyond the RP 2A bound with subsequent increases in load. The effect that tubular joint flexibility and limiting capacity may have on the overall response is allowed for in analysis V4.



Ls17u - Sec. 17.0 Ultimate Strength Load	**2702 Kips**
Reference Level Load	**2268 Kips**
Load Level at First Member Failure	**3910 Kips**
Ultimate Capacity (Ru)	**4046 Kips**
Reserve Strength Ratio	**1.784**
Platform Failure Mode	**Jacket**

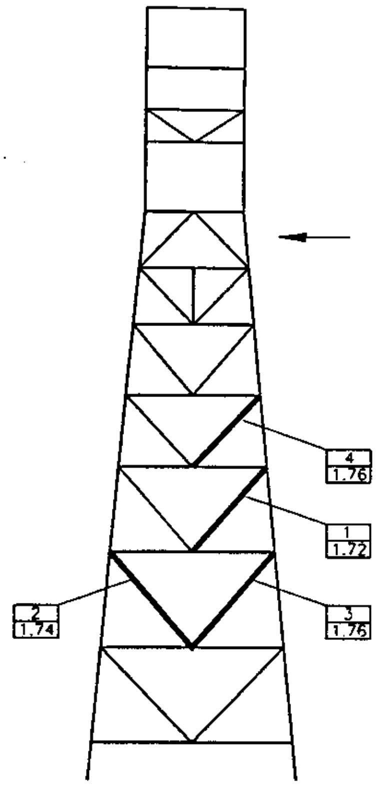
Figure 7: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V1) for wave approach direction at 270° wrt N

Lateral load level at failure of the first member is 3910 Kips

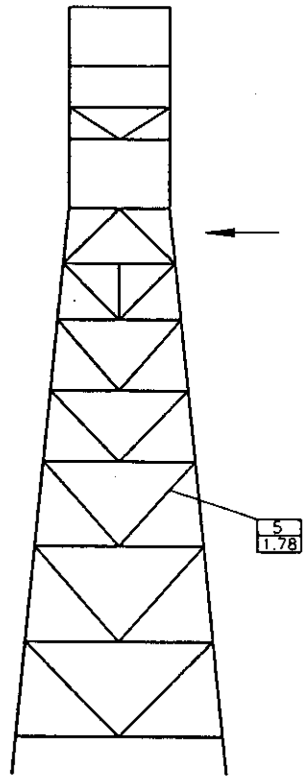
Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.533	2768		
3	0.694	3598		
4	0.756	3910	1	Buckle
5	0.764	3944	2	Yield in tension/bending
6	0.774	3989	3 / 4	Buckle
7	0.809	4046	5	Buckle
8	0.948	3799		
9	1.088	3581		
10	1.185	3470		
11	1.602	3058		

* See Figure 8 indicating location and sequence of component failures

Table 8: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V1) for wave approach direction at 270° wrt N



Platform North frame



Platform South frame

Figure 8: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case
with no joint flexibility (V1) for wave approach direction at 270° wrt N

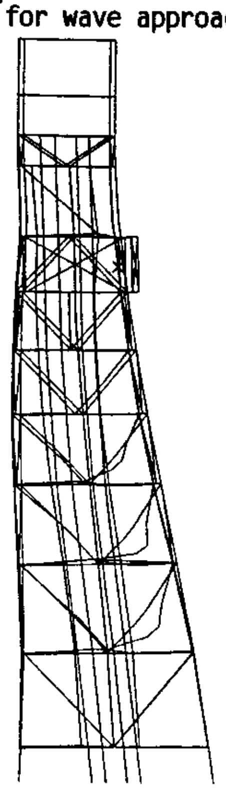


Figure 9: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case
with no joint flexibility (V1) for wave direction at 270° wrt N (mag. x 20)

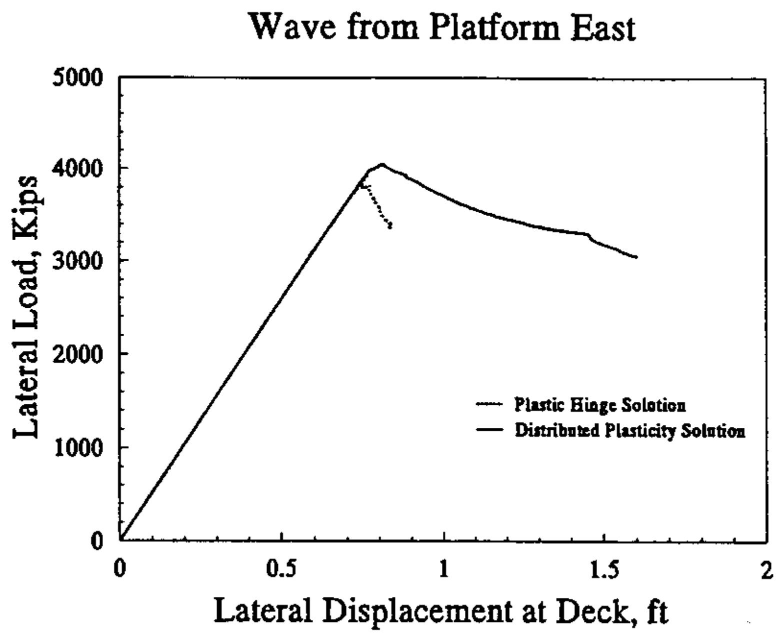
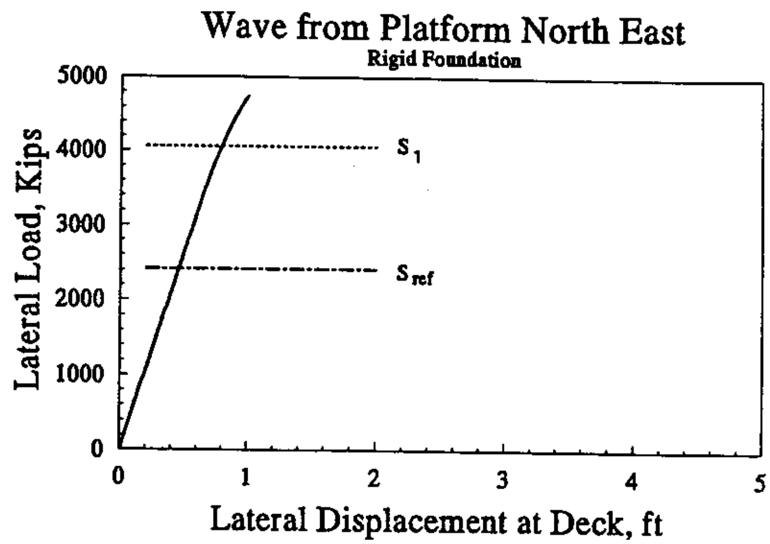


Figure 10: Ultimate Strength Analysis - Voluntary Results.
Comparison of load deflection responses for plastic hinge and distributed plasticity modelling of jacket members for rigid foundation case with no joint flexibility (V1) for wave direction at 270° wrt N

<u>Analysis V2</u>
Repeating Analysis V1 but applying the slightly higher Section 17.0 ultimate strength loads for the diagonal North East direction, gives the results shown in Figures 11 to 13 and Table 9. The same distributed plasticity model (Element Type 31) is adopted with no account of joint flexibility and with rigid foundation assumptions.

It can be seen from Figure 12 that the ultimate capacity is limited by regions of plasticity due to portal action in the legs. As the peak load is approached, plasticity develops in the thicker leg sections in the bay below failure sites 1 and 2. The distribution of the loads through all four bracing frames ensures that the members and joints are not overstressed.

At the point the analysis is stopped the Section 17.0 ultimate strength load has been sustained with a greater margin than for V1.



Ls17u - Sec. 17.0 Ultimate Strength Load	2830 Kips
Reference Level Load	2420 Kips
Load Level at First Member Failure	4057 Kips
Ultimate Capacity (Ru)	4740 Kips
Reserve Strength Ratio	1.959
Platform Failure Mode	Jacket Legs

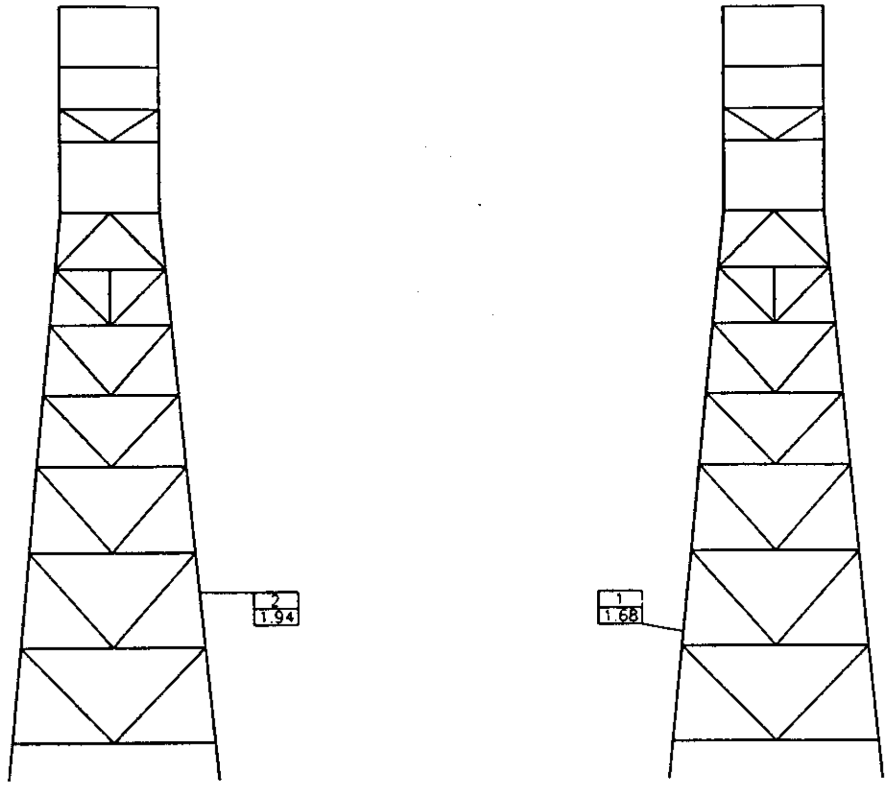
Figure 11: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V2) for wave approach direction at 225° wrt N

Lateral load level at yield of the first leg section is 4057 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.336	1754		
3	0.672	3509		
4	0.795	4057	1	Yield in compression/bend.
5	0.871	4349		
6	0.903	4453		
7	0.953	4601		
8	0.989	4708	2	Yield in tension/bending
9	1.002	4740		

* See Figure 12 indicating location and sequence of component failures

Table 9: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V2) for wave approach direction at 225° wrt N



Platform North frame

Platform South frame

Figure 12: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case
with no joint flexibility (V2) for wave approach direction at 225° wrt N

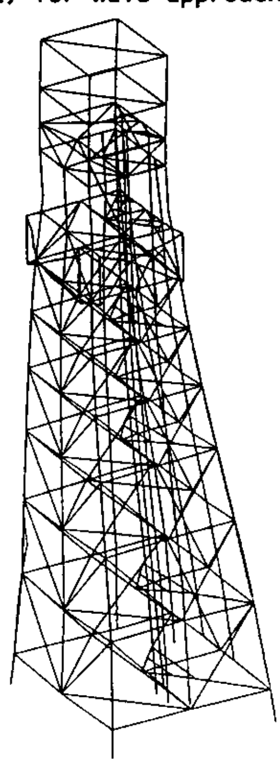
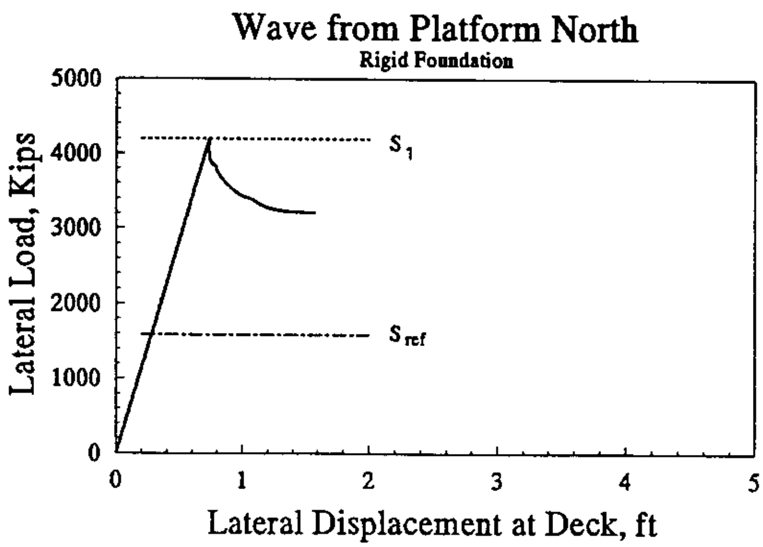


Figure 13: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case
with no joint flexibility (V2) for wave direction at 225° wrt N (mag. x 20)

Analysis V3
Repeating Analysis V1 but applying the lower Section 17.0 ultimate strength loads for the orthogonal Northerly direction gives the results shown in Figures 14 to 16 and Table 10. The same distributed plasticity model is adopted with no account of joint flexibility and with rigid foundation assumptions. An alternative plastic hinge analysis was also conducted confirming the peak capacity.

Despite the different wave/current combination and the different disposition of the conductors with respect to the incident loads, the symmetry of the primary jacket framing ensures that the load level at first member failure is very similar to the orthogonal case V1 (compare Figures 7 and 14). However the Section 17.0 ultimate strength load for this direction is significantly lower and at the point the K-braced members yield and buckle together under combined axial forces and bending (see Table 10 and Figure 15) a much higher reserve strength has been achieved. In an assessment, based on the findings for V1, it can be said that by inspection further scrutiny of the tubular joint responses is not required as this V3 analysis corresponds to a less critical direction.

Table 10 (viewed in conjunction with Figure 15) indicates that developing plasticity in both the tension and compression K-bracing between levels -126ft and -71ft has driven the automatic introduction of nonlinear distributed plasticity elements along the entire member lengths. At the peak load all components shed load together giving a rapid unloading. This contrasts with the V1 case where the stiffness gradually reduced with a sequence of component failures.



Ls17u - Sec. 17.0 Ultimate Strength Load	1880 Kips
Reference Level Load	1590 Kips
Load Level at First Member Failure	4196 Kips
Ultimate Capacity (Ru)	4196 Kips
Reserve Strength Ratio	2.639
Platform Failure Mode	Jacket

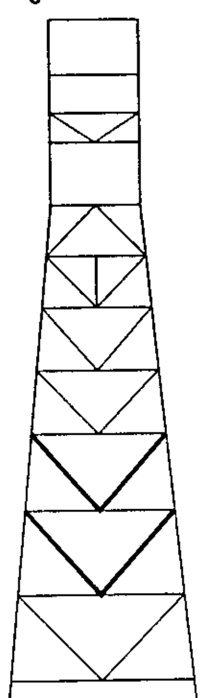
Figure 14: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N

Lateral load level at failure of the bracing is 4196 Kips

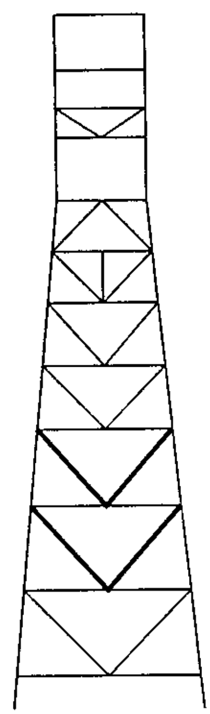
Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.264	1496		
3	0.637	3613		
4	0.743	4196	1-4 / 5-8	Yield/buckle
5	0.781	3835		
6	0.813	3732		
7	0.966	3464		
8	1.131	3326		
9	1.367	3224		
10	1.569	3214		

* See Figure 15 indicating location of component failures

Table 10: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N



Platform West frame



Platform East frame

Figure 15: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case with no joint flexibility (V3) for wave approach direction at 180° wrt N

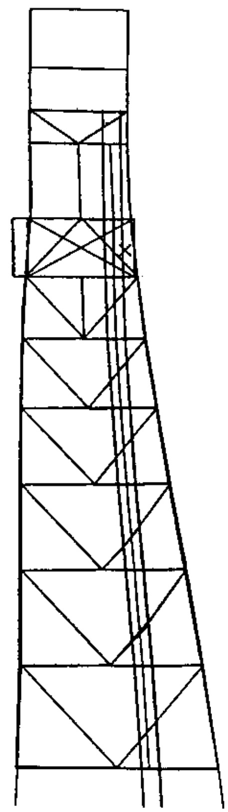


Figure 16: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case with no joint flexibility (V3) for wave direction at 180° wrt N (mag. x 20)

Analysis V4

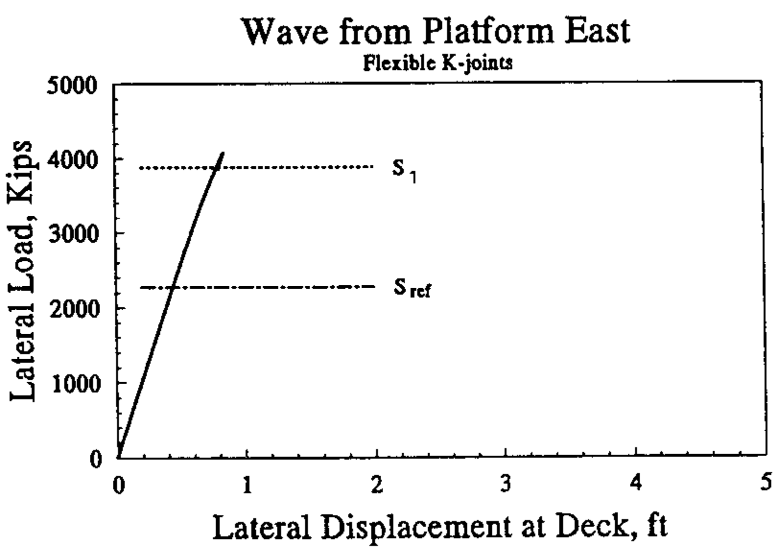
In analysis V4 the base V1 model is adopted but account is taken of the flexibility of the primary K joints between levels -126ft and -27ft as these were shown to be highly utilised in the V1 analysis. The results are presented in Figures 17 to 19 and Table 11.

The code checks to API RP 2A 20th Edition with safety factors removed demonstrated in Analysis V1 that the joint loading would reach the code lower bound before the mean strength of the members (as simulated in the analysis) was achieved. Two problems are faced: Section 17.0 is directed at a best estimate representation of system capacity with all bias removed; the joint deformations will be considerable prior to the attainment of the limiting capacity and may be limited by subsequent fracture. A direct application of the API RP 2A lower bound as a limiting capacity fails to address both these points.

In analysis V4 the nonlinear axial and in-plane and out-of-plane bending resistances at the joints is modelled with decoupled springs with 3 to 5 part linear representations of the flexibility and softening responses, as appropriate. SAFJAC analyses base the limiting capacities on the equations published in OTH 89 308 as a mean fit to the data underlying the HSE Guidance Notes strength equations. The source material for the same data forms the basis of parametric stiffness equations used within SAFJAC. Elastic flexibility limits and peak deformations are determined on the basis of tubular joint parameters, β, γ, ζ, θ etc. for ready implementation in the analysis.

The HSE equations afford significantly higher capacity to β=1.0 K joints than API and for this reason joint failures in Analysis V4 do not have a drastic influence on the response as was suggested by the RP 2A code check in Analysis V1. The HSE K joint capacity has been verified in recent experimental tests on components and frames in the JIP R&D project. In analysis V4 it is found that the joint flexibility contributes to differences in the load distribution such that a very slightly higher peak load is attained. The RSR is 1.80 in comparison with 1.78 for Analysis V1. The influence of joint flexibility is also seen in the less stiff global response as the peak load is reached. The first capacity limit is reached for a North frame K joint. However, the failure precipitating global load shedding is a member, as shown in Figure 18. This demonstrates the ability of SAFJAC to evaluate load interaction effects leading to collapse of the member separately from joint conditions. This ensures that the critical component is dictated by the analysis and load interaction effects within the structure and not the user. Allowance is made for fracture and rapid load shedding at the K joint replicating physical evidence for similar joints exposed to Hurricane Andrew and in experimental ultimate strength tests. This dictates the sharp load reduction beyond the peak global load.

Figure 18 shows the regions of member plasticity and the location of tubular joints where the capacity has been exceeded. In Figure 19 bowing of the members can be seen.



Ls17u - Sec. 17.0 Ultimate Strength Load	2702 Kips
Reference Level Load	2268 Kips
Load Level at First Joint Failure	3880 Kips
Ultimate Capacity (Ru)	4077 Kips
Reserve Strength Ratio	1.797
Platform Failure Mode	Jacket

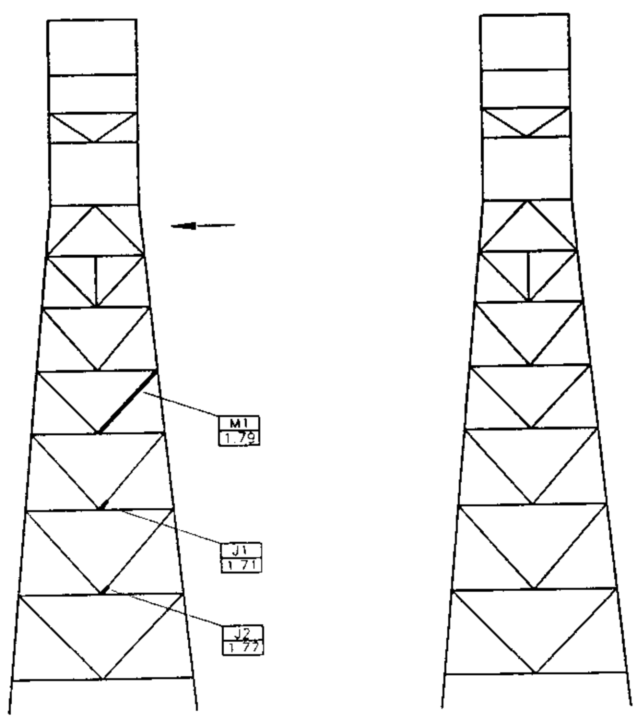
Figure 17: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N

Lateral load level at failure of the first K joint is 3880 Kips

Load Step	Disp. at +43' at SE Leg (ft.)	Lateral Load (Kips)	Element Failures*	Component Failure Mode
1	0.000	0		
2	0.435	2214		
3	0.594	3017		
4	0.794	3880	J1	K joint
5	0.822	3989		
6	0.840	4053	J2	K joint
7	0.847	4077	M1	Buckle
8	0.807	3860		
9	0.816	3890		

* See Figure 18 indicating location and sequence of component failures

Table 11: Ultimate Strength Analysis - Voluntary Results
Load displacement and failure mode results for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N



Platform North frame   Platform South frame

Figure 18: Ultimate Strength Analysis - Voluntary Results
Location and sequence of component failures for rigid foundation case with flexible joints (V4) for wave approach direction at 270° wrt N

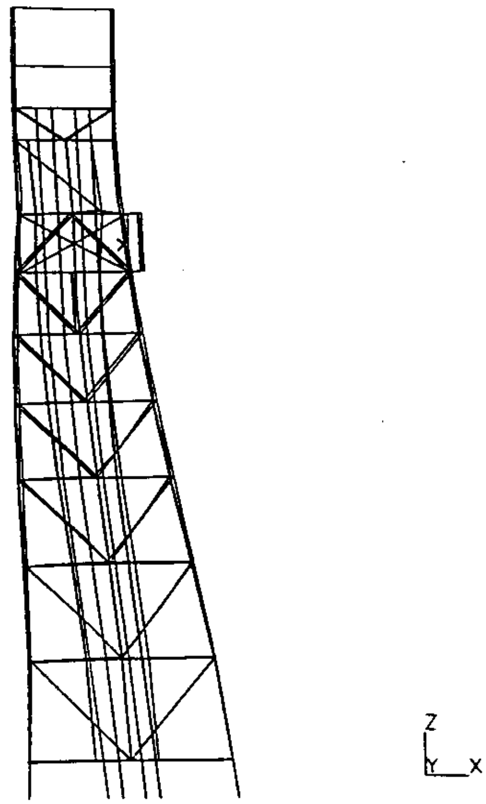


Figure 19: Ultimate Strength Analysis - Voluntary Results
Deflected shape of jacket at peak load for rigid foundation case
with flexible joints (V4) for wave direction at 270° wrt N (mag. x 20)

The series of voluntary analyses and comparison with the required case, demonstrate the influence of modelling assumptions. Figure 20 compares three results for the 270° wave attack direction. For the specific installation, the foundation compliance has a significant influence on the response. Within this structure, the joints have little influence on the ultimate strength once proper account is taken of their mean capacity but the mode of failure and post-peak response are significantly different.

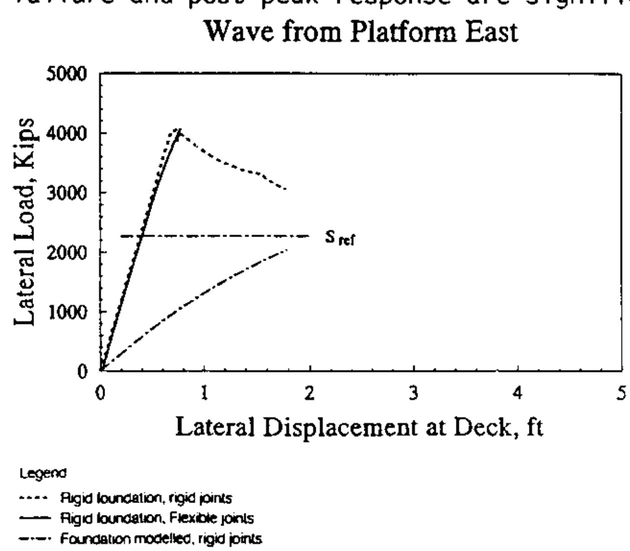


Figure 20: Ultimate Strength Analyses
Load displacement results for wave direction at 270° wrt N (mag. x 20)

PART B: REVIEW AND FEEDBACK TO THE API TG 92-5

In this section comments are made on the use of the draft Section 17.0 (April 29 version) as applied to the Gulf of Mexico benchmark structure.

Software

In 17.7.2b and 17.7.3b it is recommended that the clauses read "software developed *and validated* for that purpose".

Conductors

In the final clause of the Commentary, C17.7.3c (g), it is required that the gap between jacket and conductor be modelled. Clearly this is aimed at realism, however there is uncertainty in the initial position of the conductor in the slot. For this reason the added complexity may not necessarily lead to an improved representation of the system behaviour. Perhaps it need not routinely be modelled but if the criteria are only just met this and other factors such as initial member out-of straightness etc should be recommended for inclusion in a sensitivity study.

Ultimate strength modelling

Section 17.7.3c provides instructions on element grouping and this is expanded significantly in the commentary. It is questioned whether the level of guidance in the guideline itself is helpful. It is suggested that the clause should reiterate the intention to use best estimate properties to model components (as stated explicitly for foundations) and indicate that, if required, further guidance on the grouping of similar elements for modelling purposes is contained in the commentary.

The discussion regarding the modelling of structural members in the commentary appears to be written with the concepts of an "INTRA" type analysis in view. Other programs which have been developed and validated for ultimate strength analysis have automatic facilities to accommodate large deflection beam column action including the effects of end fixity without requiring the user to select specific K factors or element types before performing an analysis. It is also unnecessary to scrutinise working stress analysis results to establish which element types should be selected for each location "based on the dominant stresses". These software packages make the single step to ultimate strength check increasingly viable from economic and time standpoints.

Perhaps a more general approach would be to state that the modelling should properly account for beam column effects, the potential onset of plasticity, and the effect of frame restraints on buckling capacity etc. This generality leaves the analyst better able to interpret the guideline and less likely to give inadequate consideration to factors which may cursorily be disregard as irrelevant.

Deck loading

The presentation of deck loading in the commentary C17.6.2 could be open to different interpretation. For example wave loads on the net silhouette area are readily distributed equally to decks above and below. In reality structural members might share the load top to bottom whereas loads incident on equipment/structure standing on the deck will pass loads to the lower level almost exclusively. Should the net area modelling be associated with the net deck area for attracting loads rather than the between deck silhouette. Alternatively the proposed procedure may be adequate but should perhaps be flagged for further investigation in a sensitivity study should the margin beyond the required ultimate strength be small.

Typographical

17.3.1c "platform *is* not"

17.5.2 "environmental" - remove space and hyphen

17.6.2b "Section 17.6.2a.2" ? There is no 17.6.2a.5

17.7.3 " to ensure adequacy"

17.7.3c "deformation"

For clarity

In 17.2.6 suggest changing word "total" to "combined effect".

In 17.5.4 "API RP 2N First Edition" is referenced several times, but never typed the same way.

ANNEX

SUPPLEMENTARY ANALYSIS SUBSEQUENT TO ISSUE OF PMB DRAFT REPORT

(C6350R01.18 Rev 0 November 1994)

1. INTRODUCTION

Having examined the draft benchmark report issued by PMB, Participant K undertook further investigation of two aspects. Although the hydrodynamic parameters, and therefore the load distributions, derived from the Section 17/API RP 2A 20th Edition loading philosophies differed, the range in ultimate strengths calculated was unexpected given the dominance of the foundation. API RP 2A gives a range for the residual T-Z capacity and in the submitted analyses the lower $T_{res}=0.7T_{max}$ value had been adopted. An additional analysis was therefore undertaken for the critical diagonal wave attack direction (225° with respect to platform North) in which the upper plateau value of 0.9T was adopted. The result and comparison with the original results is presented in Section 2, demonstrating the strong dependence of this platform's ultimate strength on the T-Z resistance modelling.

The difference in elastic stiffness for the fixed base analyses reported in the PMB draft report were remarkable. Comparison of fixed and pinned base conditions is made in Section 3, but demonstrate that other modelling factors are responsible for the reported differences.

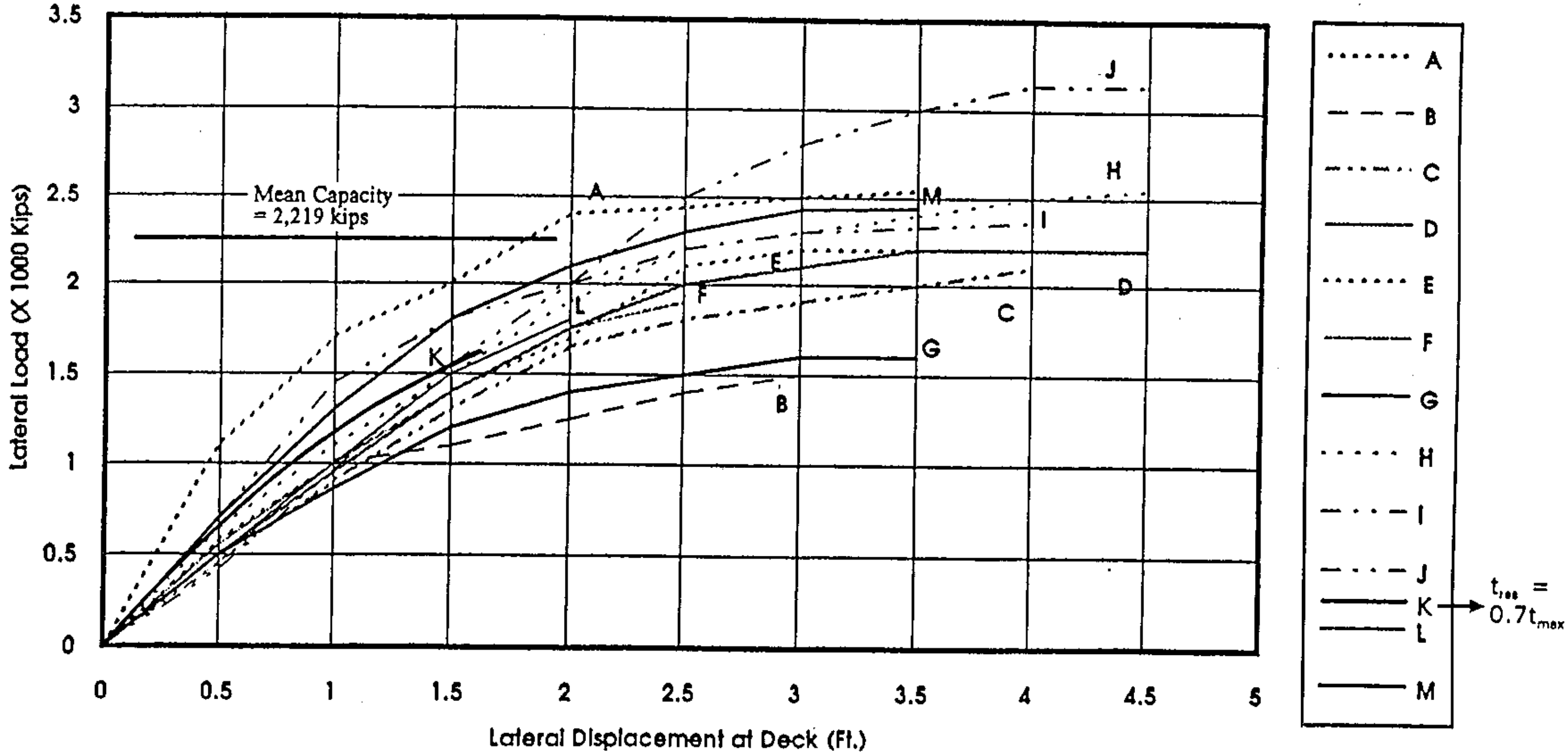
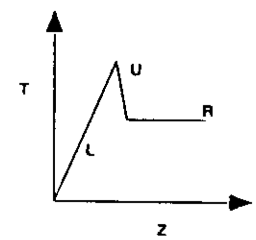


Figure 1: Load displacement behaviour for diagonal wave attack direction (taken from PMB draft final report) Analysis K adopts $T_{res}=0.7T_{max}$ for clays

2. FOUNDATION RESISTANCE

The results for the required analysis performed by Participant K for the diagonal wave attack direction (225° with respect to platform North) are highlighted on the figure taken from the draft PMB report in Figure 1. The capacity was limited by the T-Z foundation resistance as indicated in Table 1 taken from the Participant K report. The residual capacity was set at the API RP 2A lower bound, $T_{res}=0.7T_{max}$.

Spring	T_residual (Kips)	Pile - 1 (Kips)	Pile - 2 (Kips)	Pile - 3 (Kips)	Pile - 4 (Kips)
1	13.08	-13.36 R	-18.37 L	-18.35 L	13.36 R
2	13.08	-13.36 R	-17.97 L	-17.94 L	13.36 R
3	13.08	-13.36 R	-17.58 L	-17.55 L	13.36 R
4	13.08	-13.36 R	-17.04 L	-16.95 L	13.36 R
5	28.10	-28.11 R	-33.92 L	-33.74 L	28.11 R
6	28.10	-28.11 R	-32.54 L	-32.37 L	28.11 R
7	28.10	-28.11 R	-31.27 L	-31.10 L	28.11 R
8	28.10	-28.11 R	-30.07 L	-29.91 L	28.11 R
9	42.19	-42.24 R	-43.27 L	-43.05 L	42.24 R
10	42.19	-42.24 R	-41.16 L	-40.94 L	42.24 R
11	42.19	-42.24 R	-38.78 L	-38.59 L	42.24 R
12	42.19	-42.24 R	-36.25 L	-36.08 L	42.24 R
13	56.07	-56.05 R	-45.09 L	-44.88 L	56.05 R
14	56.07	-56.05 R	-41.88 L	-41.69 L	59.70 U
15	56.07	-56.05 R	-37.83 L	-37.50 L	64.22 U
16	56.07	-56.05 R	-32.41 L	-32.13 L	68.55 U
17	71.11	-71.10 R	-34.84 L	-34.54 L	92.20 U
18	71.11	-71.10 R	-29.32 L	-29.06 L	97.12 U
19	71.11	-71.10 R	-24.38 L	-24.17 L	101.44 L
20	71.11	-71.10 R	-19.93 L	-19.76 L	94.44 L
21	224.00	-224.00 R	-71.63 L	-71.01 L	224.00 R
22	224.00	-224.00 R	-53.90 L	-53.44 L	224.00 R
23	299.45	-323.51 U	-32.35 L	-32.07 L	244.57 L
24	299.45	-385.60 U	-19.96 L	-19.79 L	172.62 L
25	299.45	-426.83 L	-13.13 L	-13.01 L	115.42 L
26	299.45	-391.29 L	-9.94 L	-9.86 L	90.31 L
27(E.B.)	89.07	-34.80 L	-1.52	-1.51 L	0.00
Total	2577.07	-2853.47	-826.33	-820.99	2039.48



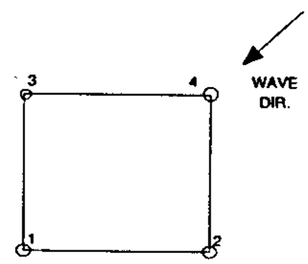


Table 1: Ultimate Strength Analysis - Required Results - $T_{res}=0.7T_{max}$
Foundation T-Z capacity for piled foundation case
with no joint flexibility (R2) for wave approach direction at 225° wrt N

It can be seen from Figure 1 that there is a range in capacities but the report also indicated that the failures were generally found to have been governed by the foundation. The Participant K result was lower than the 'mean' capacity prediction. Given the mobilisation of deformations into the residual T-Z strength regime (Table 1) an additional analysis was performed adopting the upper band allowed by API RP 2A for clays, ie. $T_{res} = 0.9\ T_{max}$.

The global load deflection response is shown in Figure 2 in comparison with the original result. The increase in residual T-Z resistance gives some 27% additional capacity to the platform. This can be seen by comparing the detailed pile results from Table 1 with Table 2 which applies to the new analysis. The small slope prescribed to the residual T-Z plateau for numerical stability means that the residual resistance appears to slightly exceed the nominal API RP 2A value. This is relatively insignificant and the analysis serves to demonstrate the dependence of the prediction on the modelling of the foundation for this platform. The result for the higher plateau T-Z is shown with the body of results in Figure 3. The results show that for Pile 1 the T-Z limiting strength is achieved as failure is about to occur due to the end bearing.

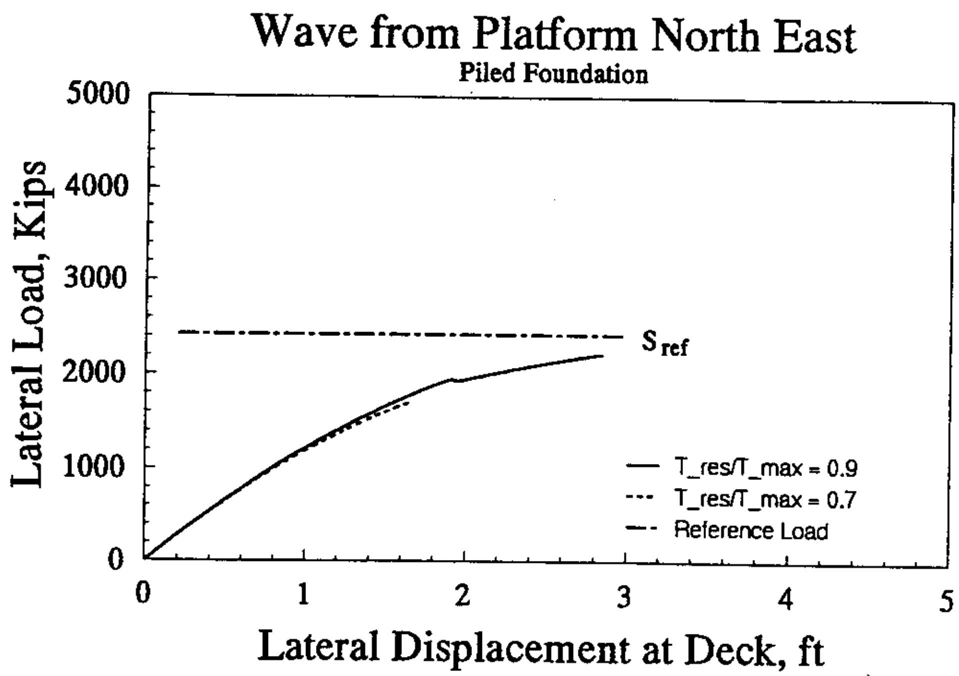
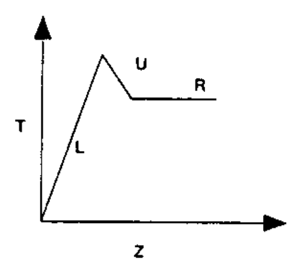


Figure 2: Ultimate Strength Analysis- Wave direction at 225° wrt Platform N
Comparison of capacities for different T-Z residual resistance
assumptions derived from API RP 2A

Spring	T_max (Kips)	T_res (Kips)	Pile-1 (Kips)	Pile-2 (Kips)	Pile-3 (Kips)	Pile-4 (Kips)
1	18.72	16.85	-21.56	-17.57	-17.56	18.80
2	18.72	16.85	-21.48	-17.78	-17.77	18.75
3	18.72	16.85	-21.40	-17.98	-17.98	18.70
4	18.72	16.85	-21.32	-18.16	-18.17	18.64
5	40.13	36.12	-40.54	-39.28	-39.29	37.88
6	40.13	36.12	-40.48	-39.57	-39.58	37.82
7	40.13	36.12	-40.41	-39.82	-39.83	37.76
8	40.13	36.12	-40.35	-40.05	-40.06	37.69
9	60.27	54.24	-58.40	-59.04	-58.96	55.75
10	60.27	54.24	-58.32	-56.96	-56.88	55.67
11	60.27	54.24	-58.23	-54.61	-54.54	55.58
12	60.27	54.24	-58.12	-50.78	-50.67	55.47
13	80.10	72.09	-75.83	-62.46	-62.33	73.18
14	80.10	72.09	-75.72	-57.19	-57.08	73.07
15	80.10	72.09	-75.61	-52.37	-52.27	72.96
16	80.10	72.09	-75.50	-47.93	-47.85	72.85
17	101.59	91.43	-94.76	-55.58	-55.49	92.11
18	101.59	91.43	-94.66	-50.95	-50.87	92.01
19	101.59	91.43	-94.56	-42.54	-42.41	91.91
20	101.59	91.43	-94.47	-34.78	-34.67	91.82
21	224.00	224.00	-227.54	-125.00	-124.59	224.89
22	224.00	224.00	-227.43	-94.07	-93.76	224.78
23	427.79	385.01	-387.59	-56.45	-56.27	385.45
24	427.79	385.01	-387.42	-34.84	-34.72	406.42
25	427.79	385.01	-387.30	-22.91	-22.83	420.60
26	427.79	385.01	-387.23	-17.35	-17.29	427.75
27	89.07	89.07	-82.67	-2.65	-2.64	0.04
Total	3451.47	3160.03	-3248.89	-1208.68	-1206.36	3198.32



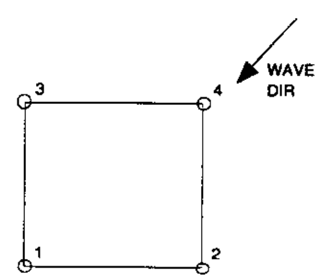


Table 2: Ultimate Strength Analysis - $T_{res}=0.9T_{max}$
Foundation T-Z capacity for piled foundation case with
no joint flexibility for wave approach direction at 225° wrt N

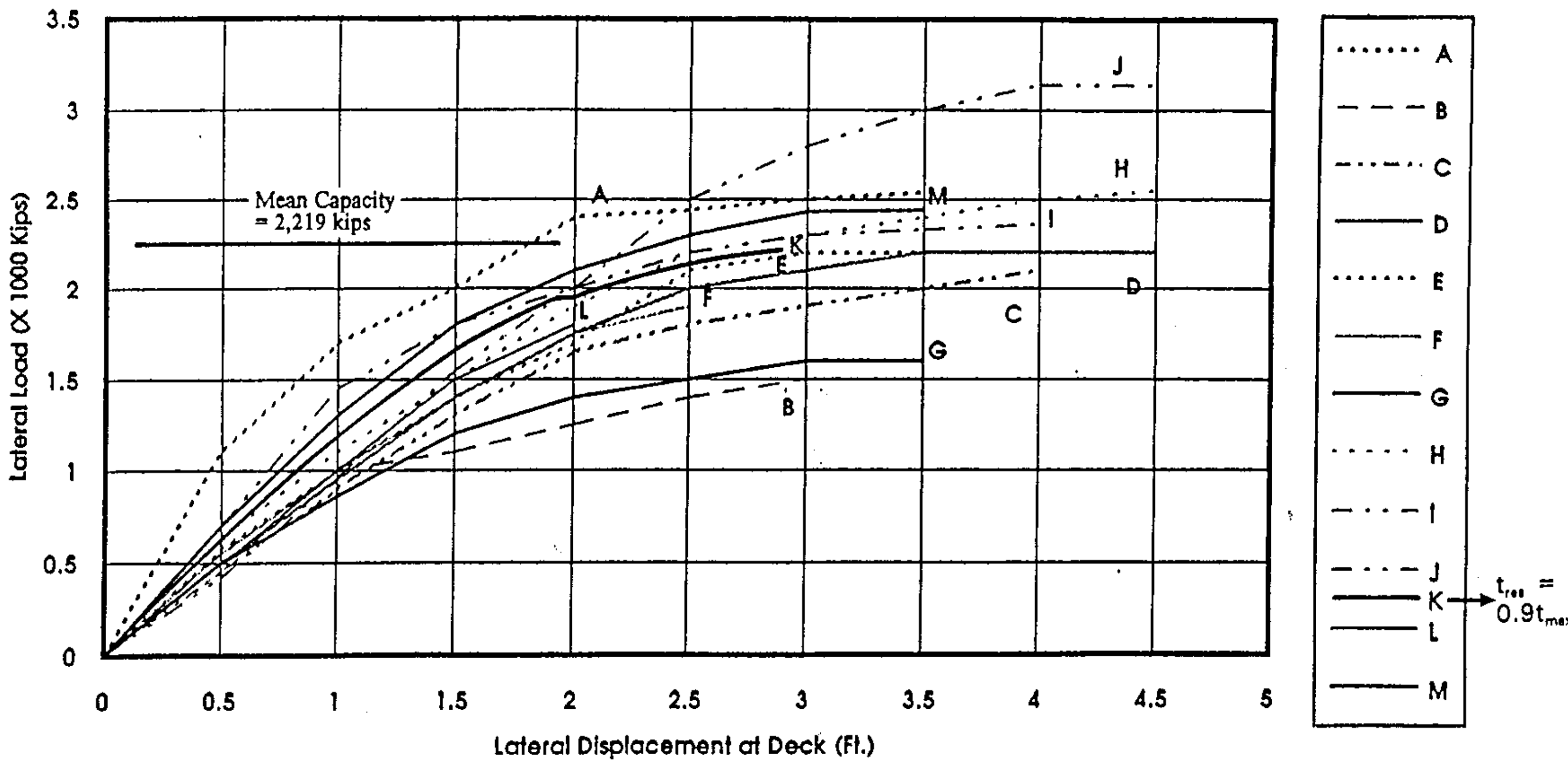


Figure 3: Load displacement behaviour for diagonal wave attack direction (225° wrt platform North)
Analysis K adopts $T_{res} = 0.9\ T_{max}$ for clays

3. STRUCTURE FIXITY

Figure 4, taken from the PMB draft report indicates the range in responses (and even in elastic stiffnesses) calculated for the 'fixed-base' case, for the wave attack direction at 270° with respect to platform North. Participant K fixed the base for the voluntary analyses some 12 feet below the mudline representing the inflection of the piles in more comprehensive analysis. All six degrees of freedom were fixed. An additional analysis was performed with pinned fixity allowing rotations but no linear deflections, to investigate the additional flexibility this may impart. Changing from the fully fixed to pinned base models had a marginal effect on the stiffness as shown in Figure 5 and the reasons for the disparity in Figure 4 remains to be explained. It is considered that omission of the piles both in the foundation and up the legs may contribute to the significantly reduced elastic stiffness for some analyses.

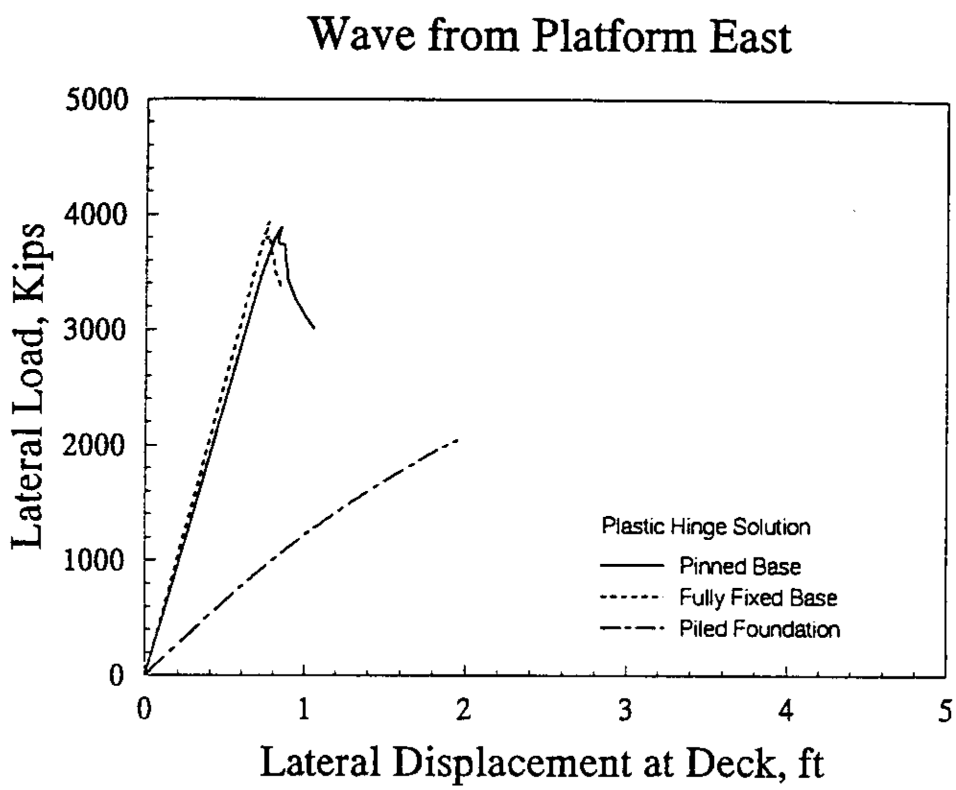


Figure 5: Ultimate Strength Analysis
Comparison of load displacement responses for different base fixity assumptions for wave approach direction at 270° wrt platform N

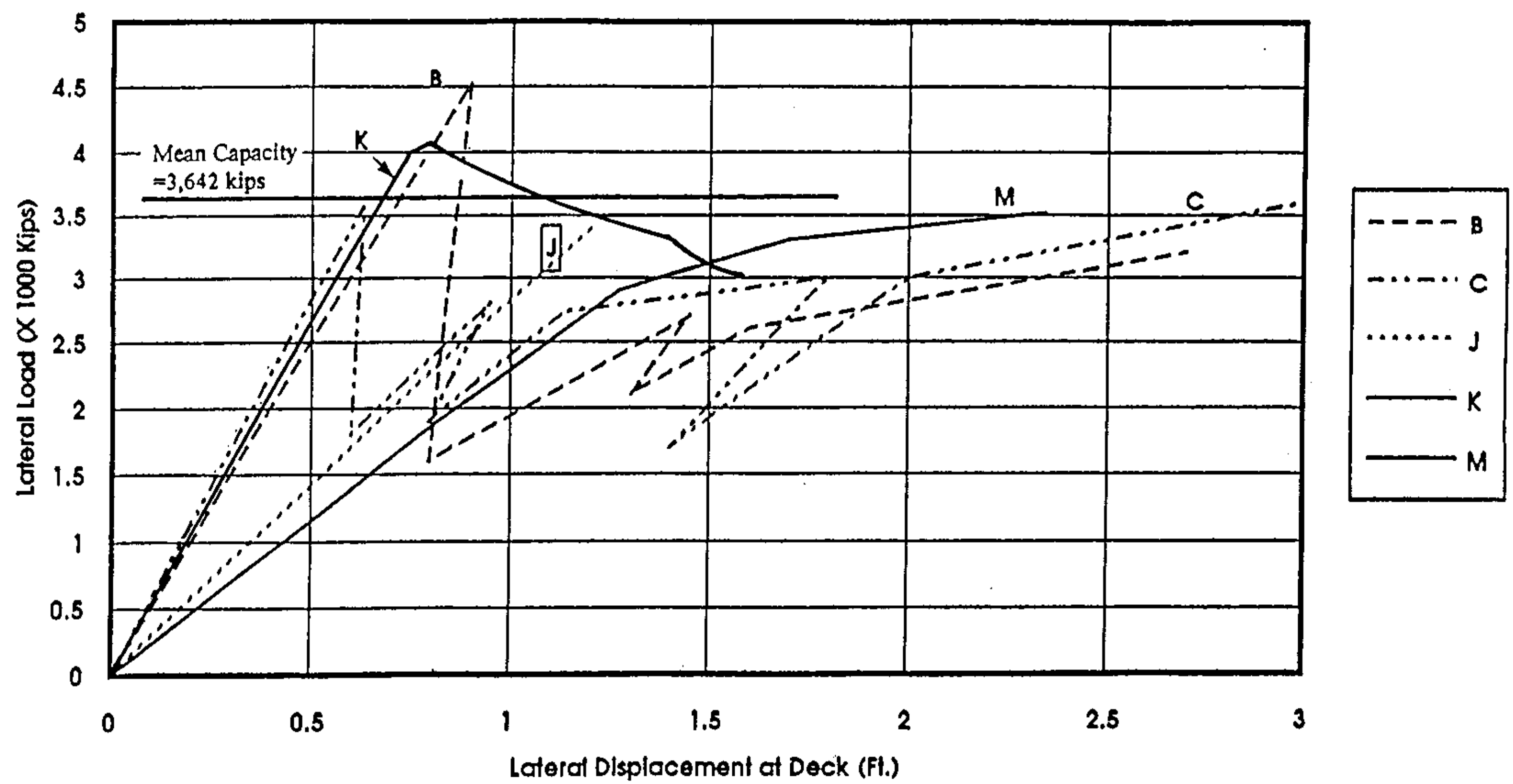


Figure 4: Load displacement behaviour for diagonal wave attack direction
(270° wrt platform North)
(taken from PMB draft final report)