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.

December 1994

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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

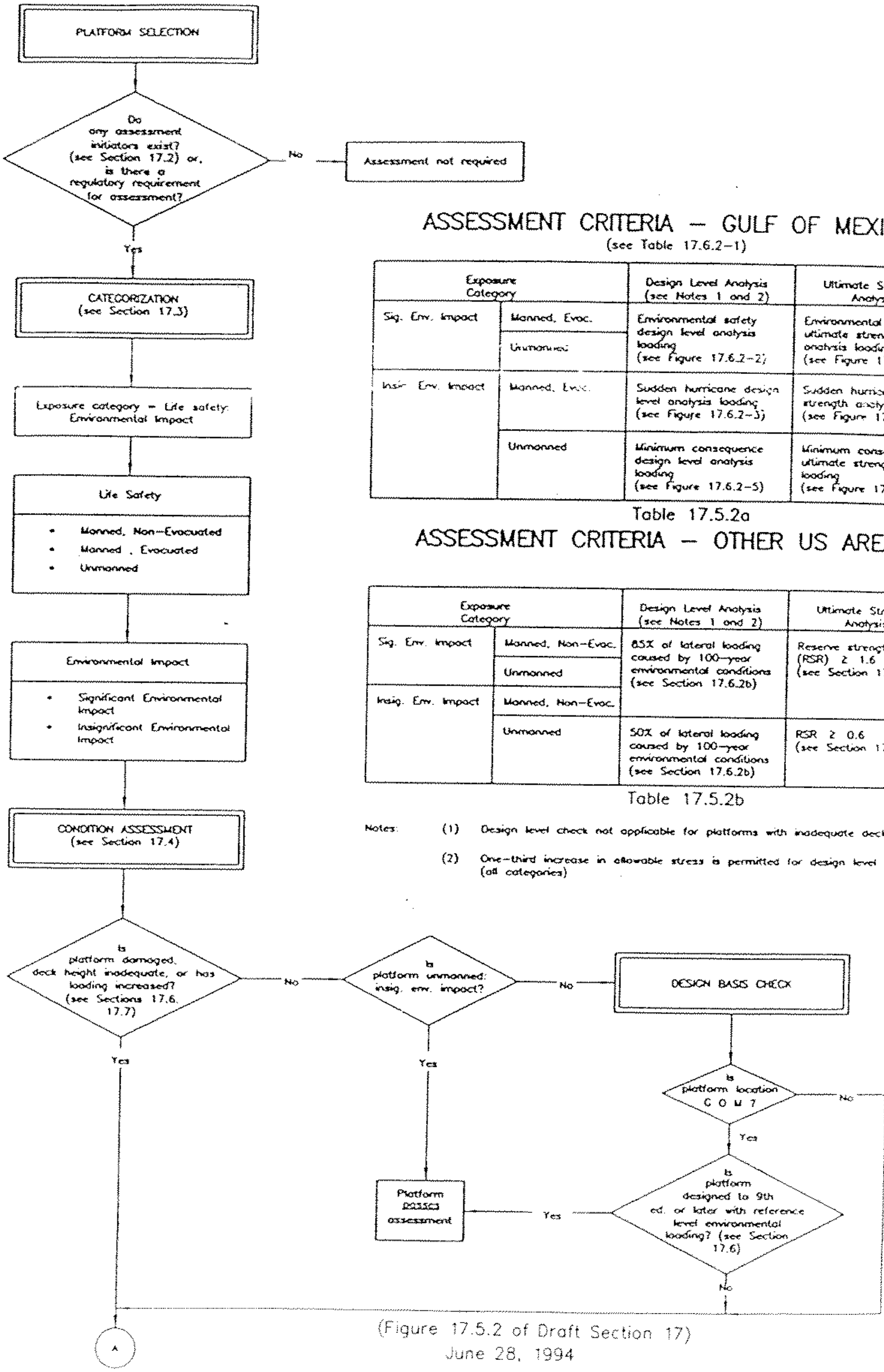
1.3 PROJECT PARTICIPANTS

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)

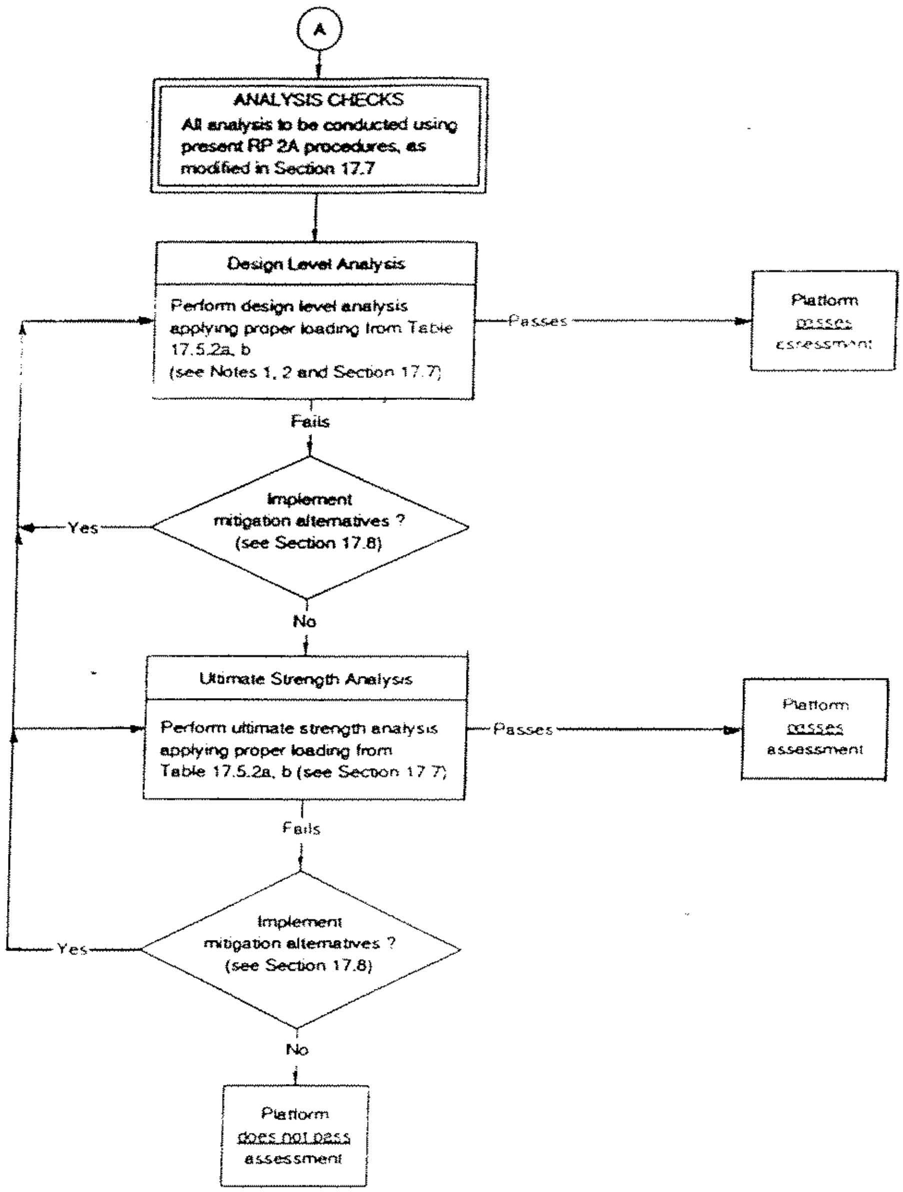
Table 17.5.2b

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.

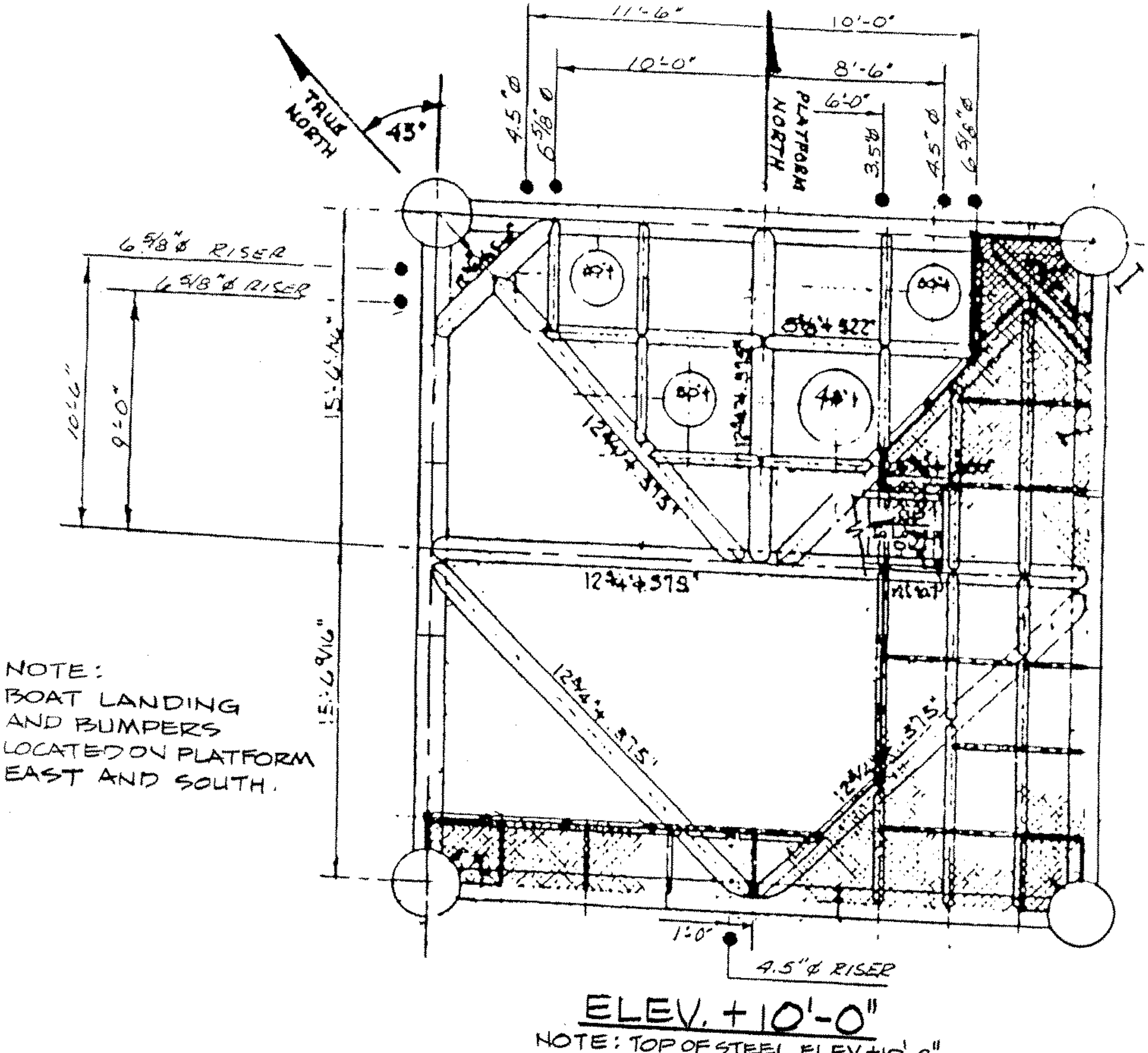


Figure 2-1 Key Plan - Benchmark Platform

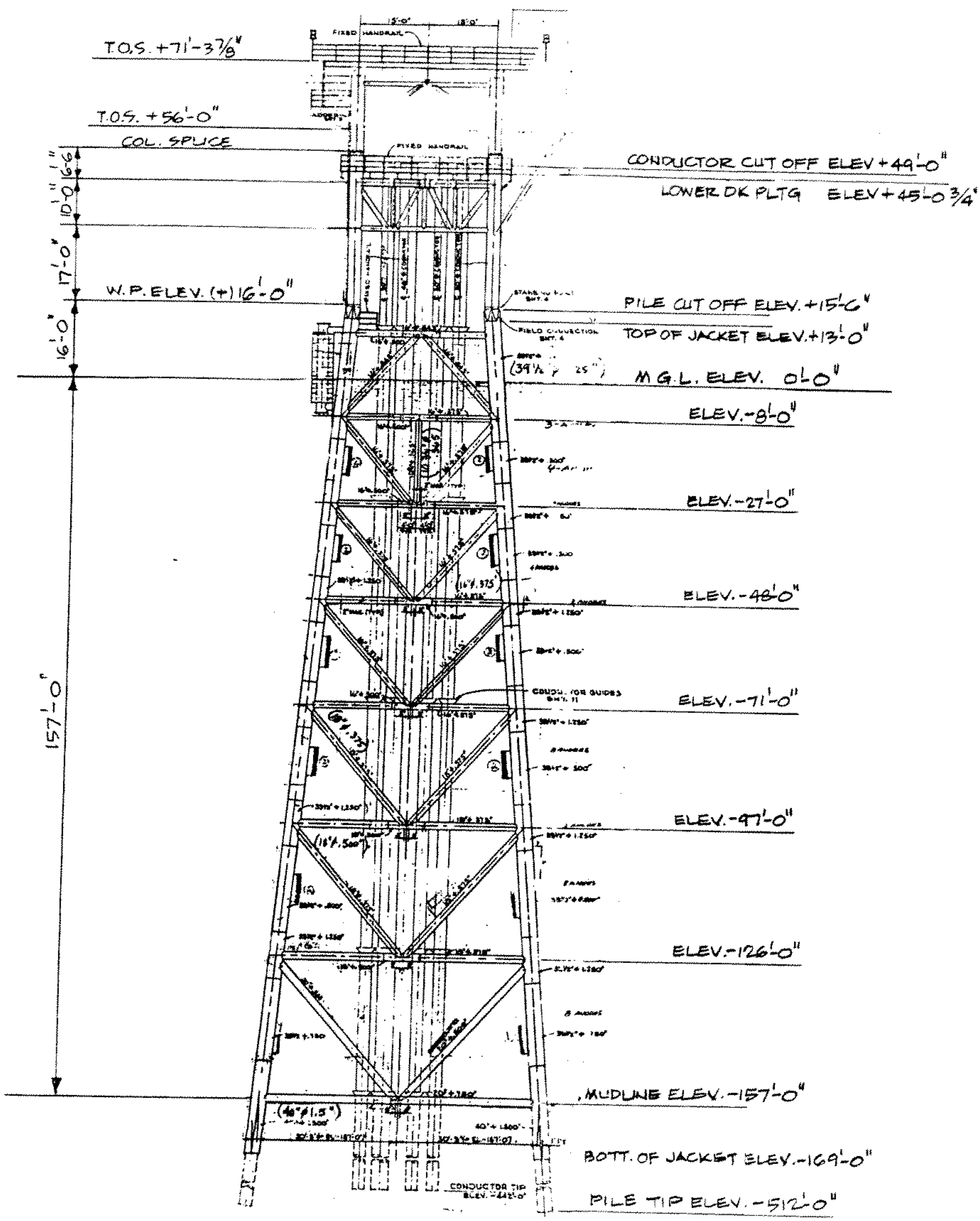
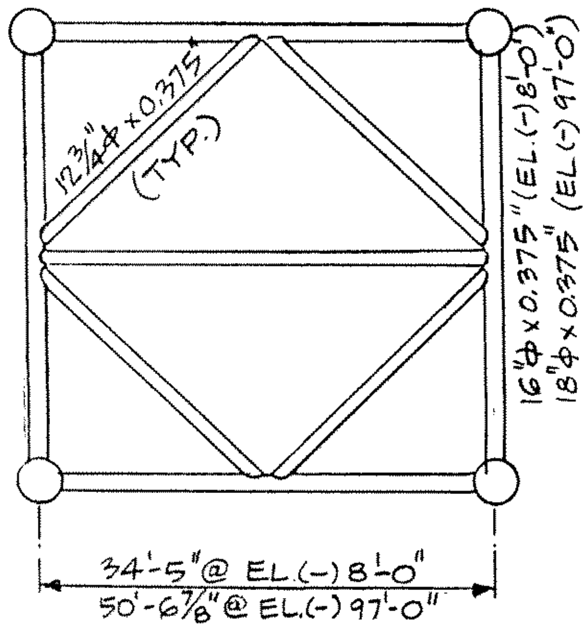
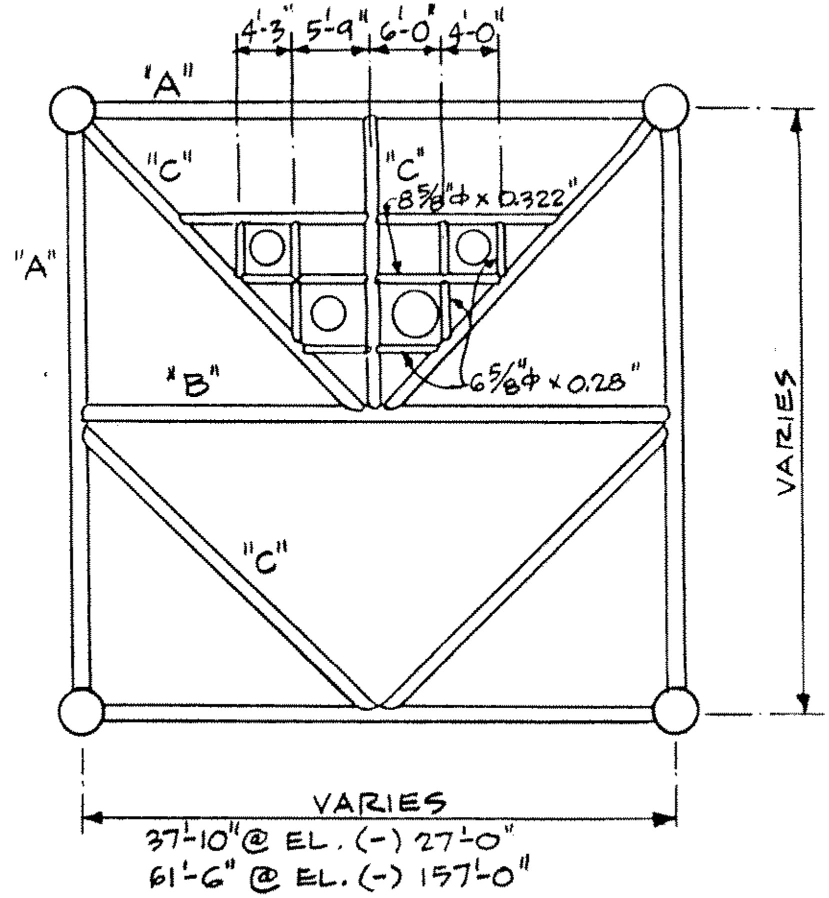


Figure 2-2 Typical Elevation - Benchmark Platform



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"	16Φ x 0.375"	14"Φ x 0.375"	12 3/4"Φ x 0.375"
(-) 48'-0'	16"Φ x 0.375"	14"Φ x 0.375"	12 3/4"Φ x 0.375"
(-) 71'-0"	16"Φ x 0.375"	16"Φ x 0.375"	12 3/4"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

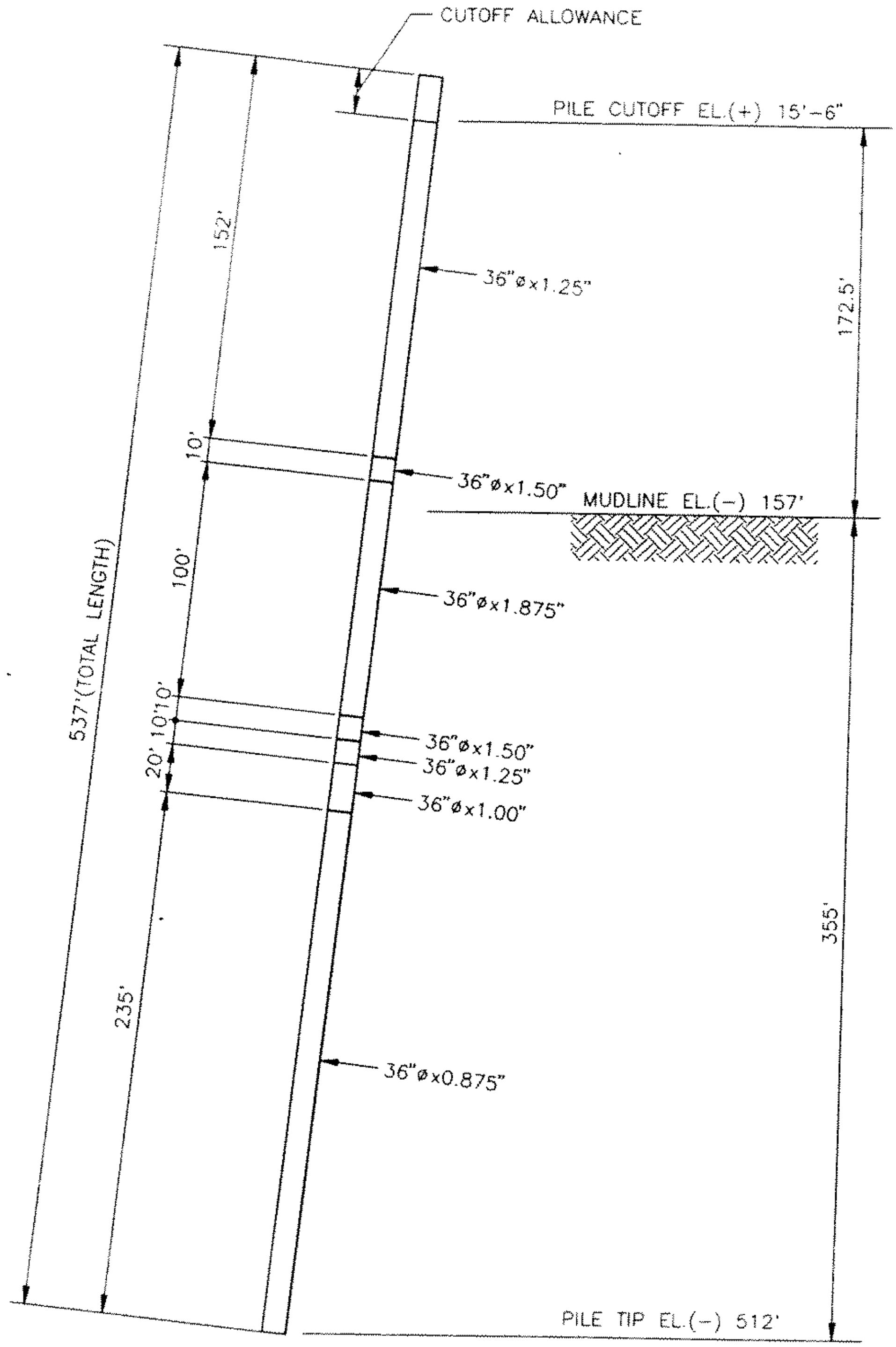


Figure 2-4 Pile Details - Benchmark Platform

-4 AH 08-31-94 PLOT SCALE 1=900 88967-315

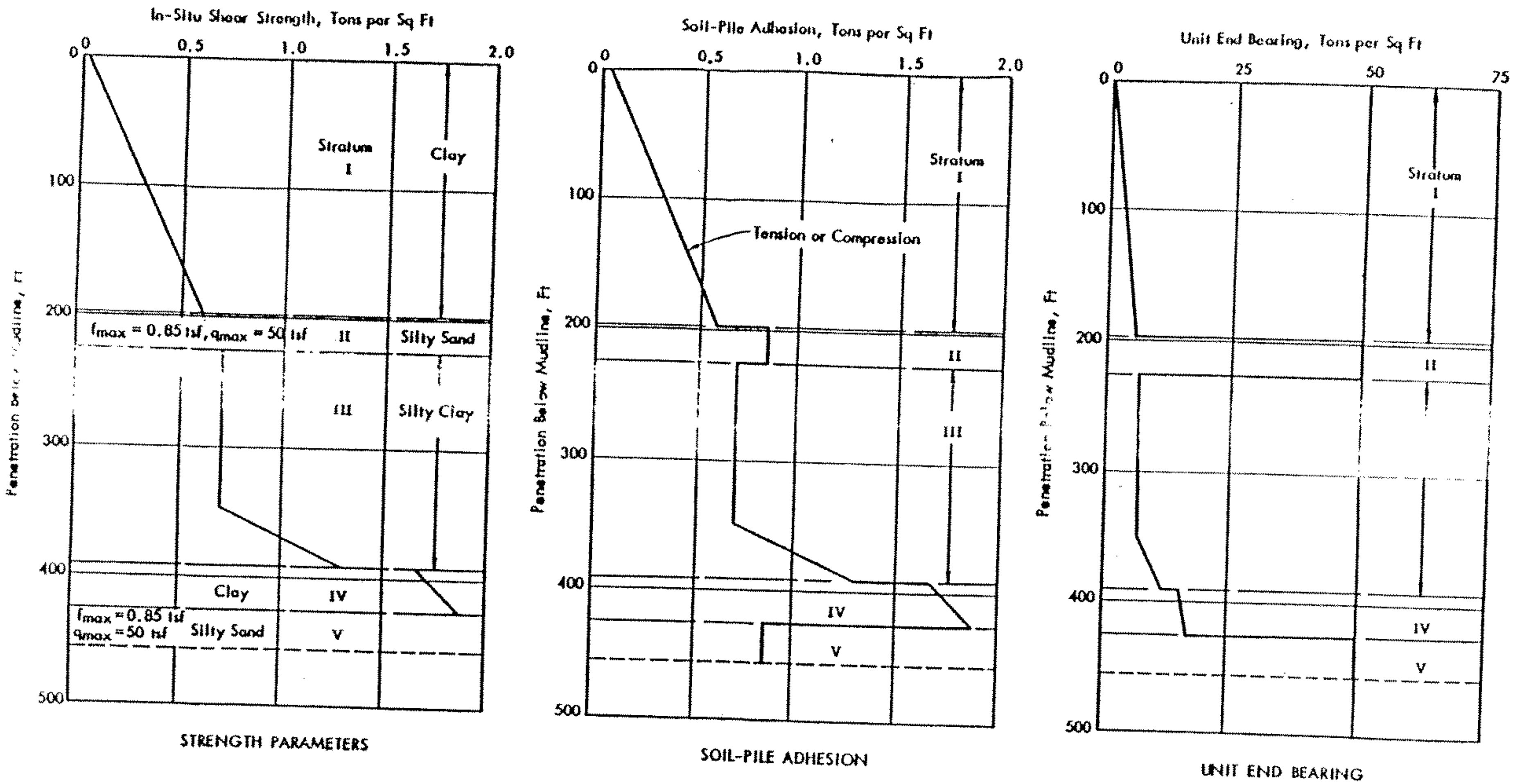


Figure 2-5 Soil Strength Parameters - Benchmark Platform

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

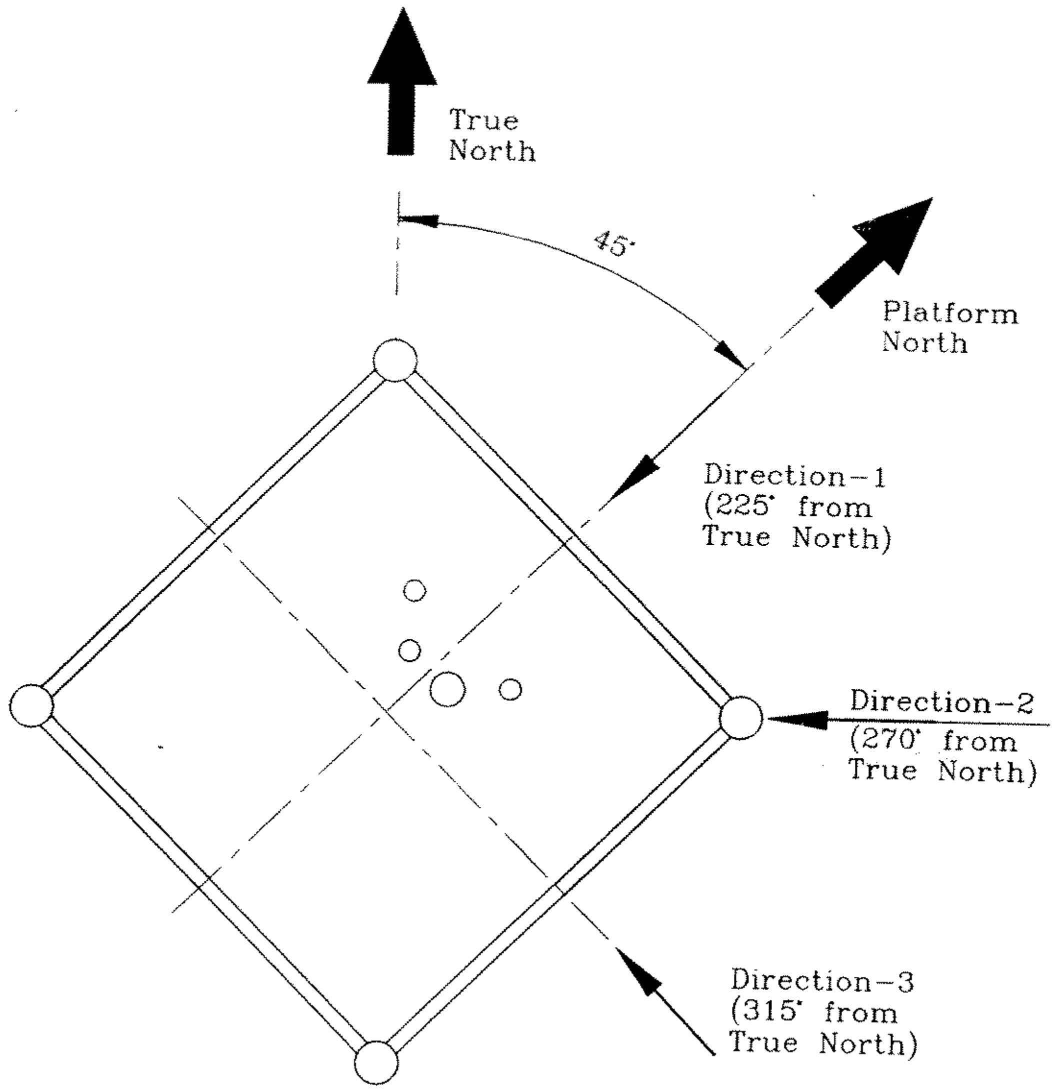
Table 3-8 and Figures 3-17 to 3-19 summarize variations in the metocean parameters, base shear, ultimate capacity, and RSR values for the three approach directions for the base case with pile/soil interaction included. These figures indicate significant variations in values obtained by the participants. 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.

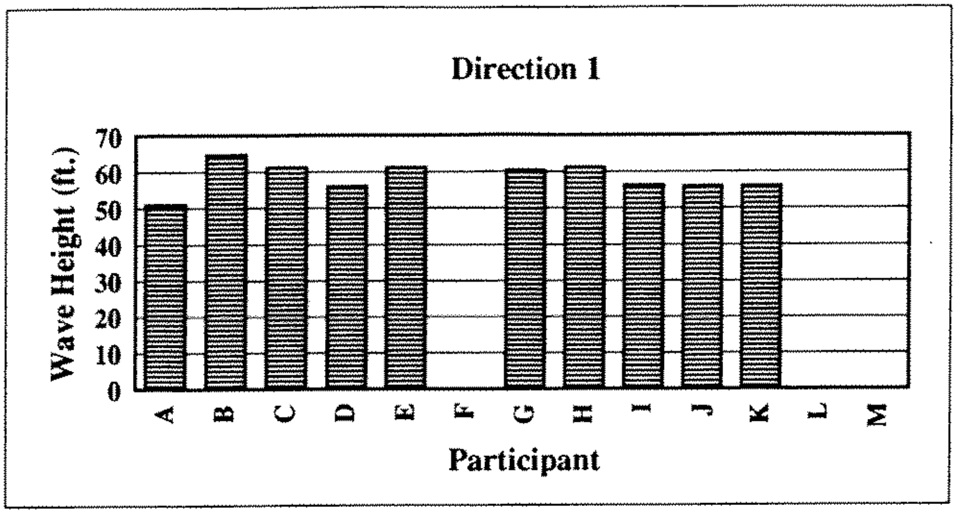
Figure 3-20(a) indicates VL (< 1,500 kips) base shear values estimated by participants A and D and VH (>3,000 kips) values by participant M. Participant G values are represented as "High" and participant F values as "Low" in this figure. The values per the other seven participants are in the "Medium" 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.

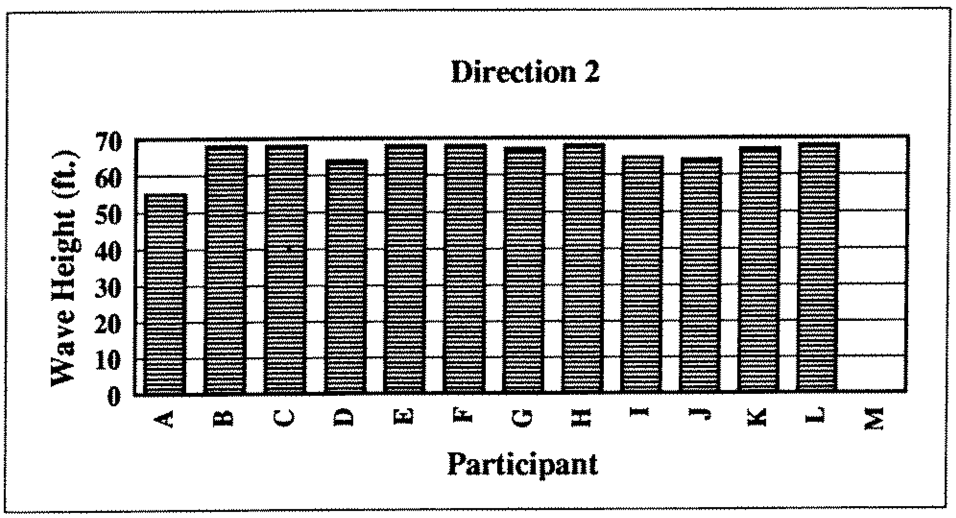


NOTE: The above three directions are basic directions referred to in the tables and figures. Tables 3-1 to 3-3 indicate normalized directions (with respect to True North) used in participants submittals.

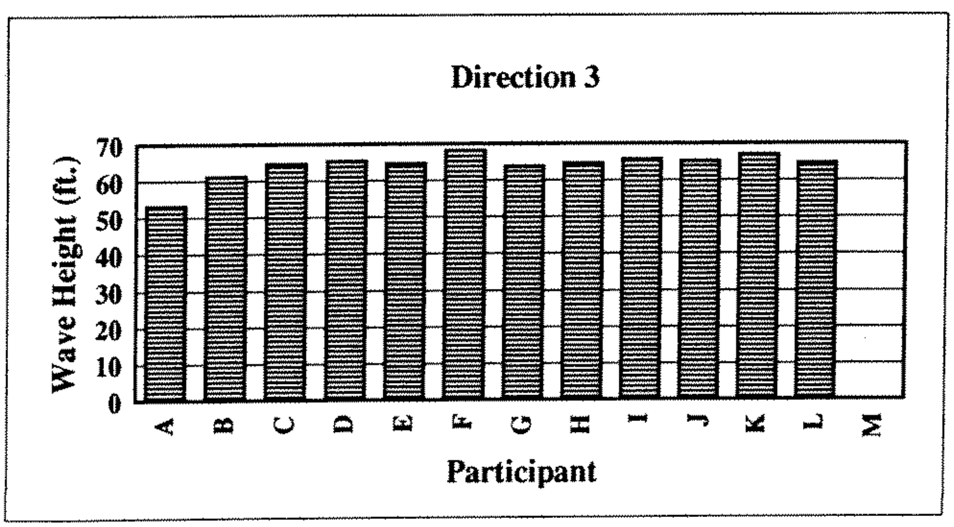
Figure 3-1 Wave Approach Directions



Mean	58.31
COV	0.07

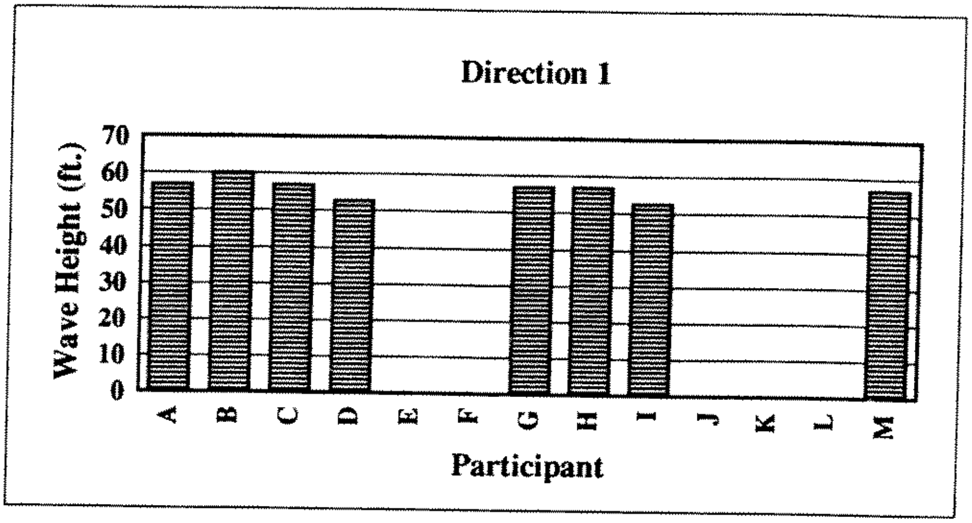


Mean	65.77
COV	0.06

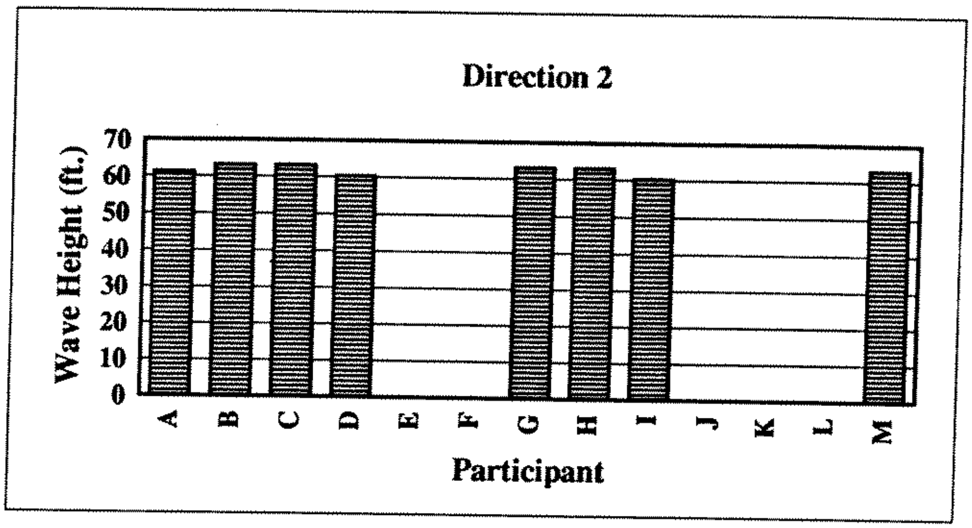


Mean	63.92
COV	0.06

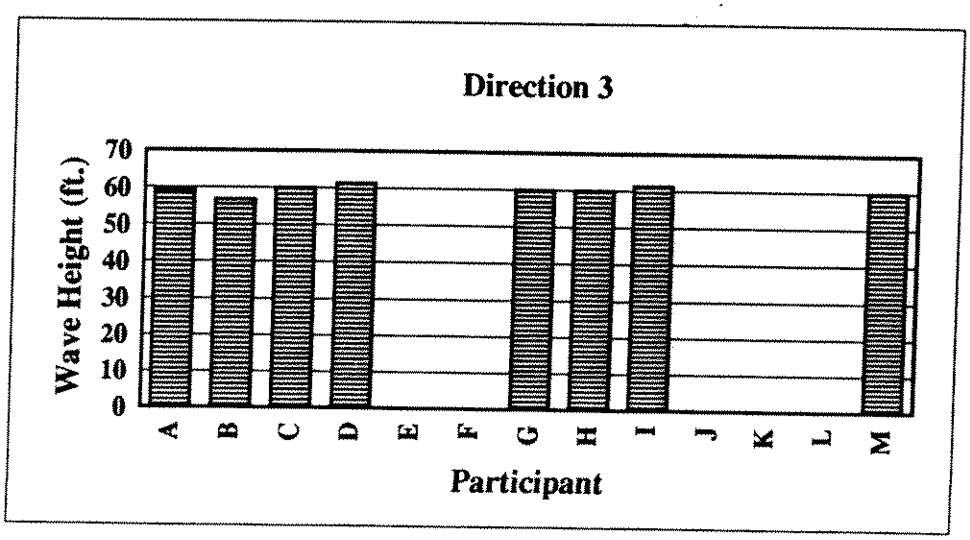
Figure 3-2: Comparison of Wave Height - Section 17 Ultimate Strength



Mean	56.04
COV	0.04

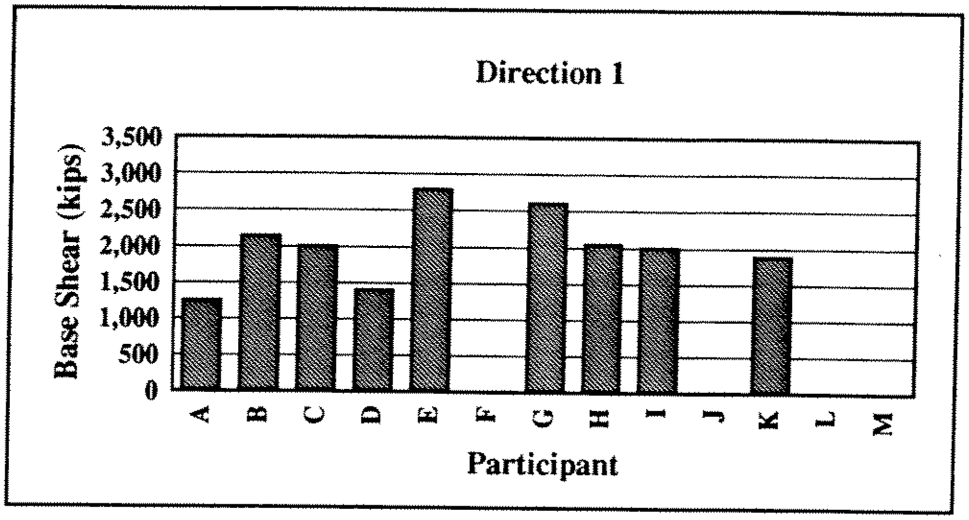


Mean	62.07
COV	0.02

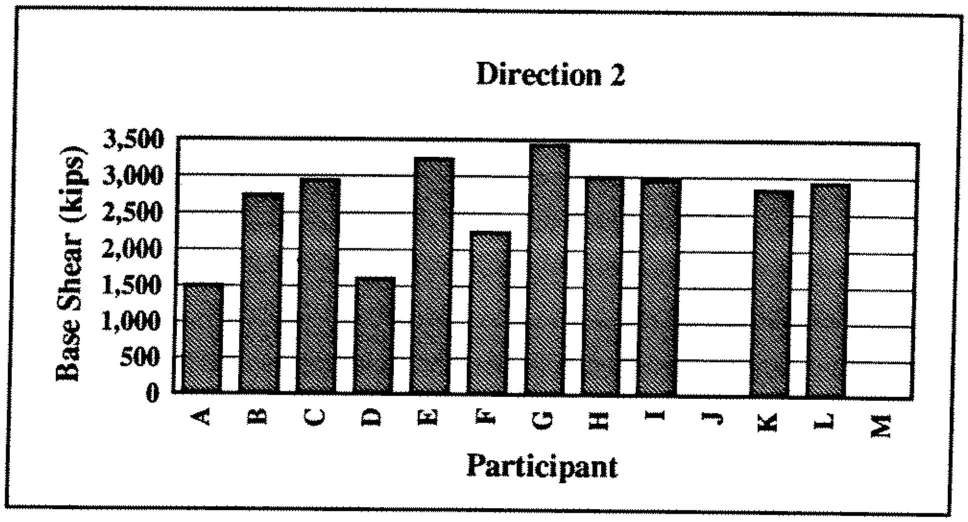


Mean	59.74
COV	0.02

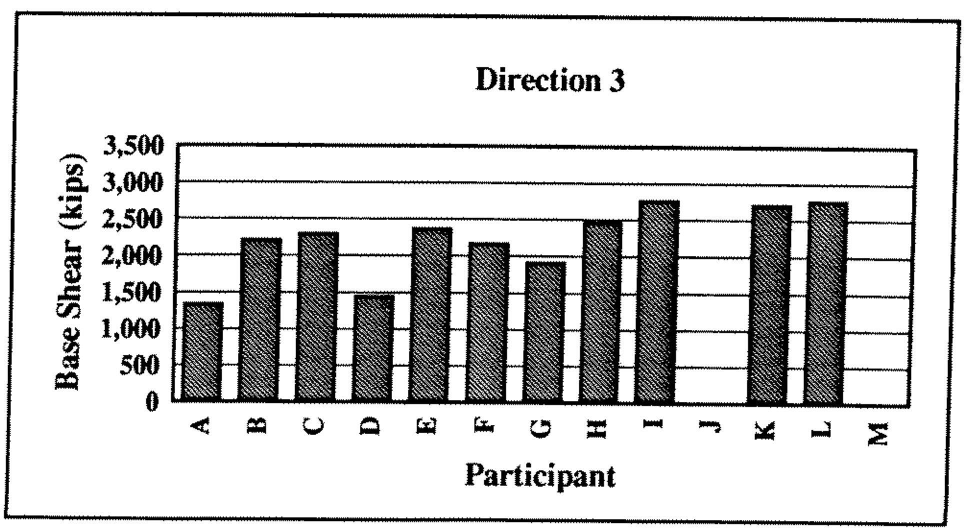
Figure 3-3: Comparison of Wave Height - RP 2A, 20th Edition



Mean	2,007
COV	0.25

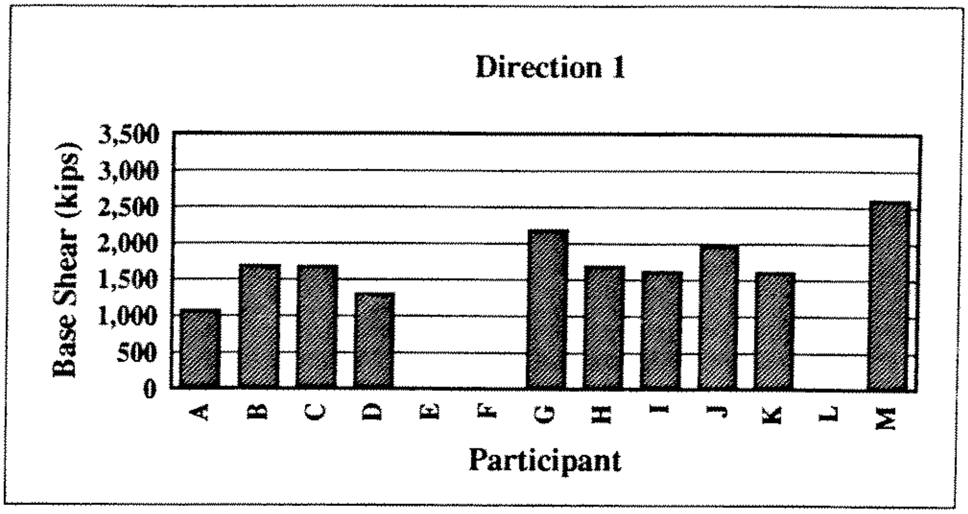


Mean	2,667
COV	0.24

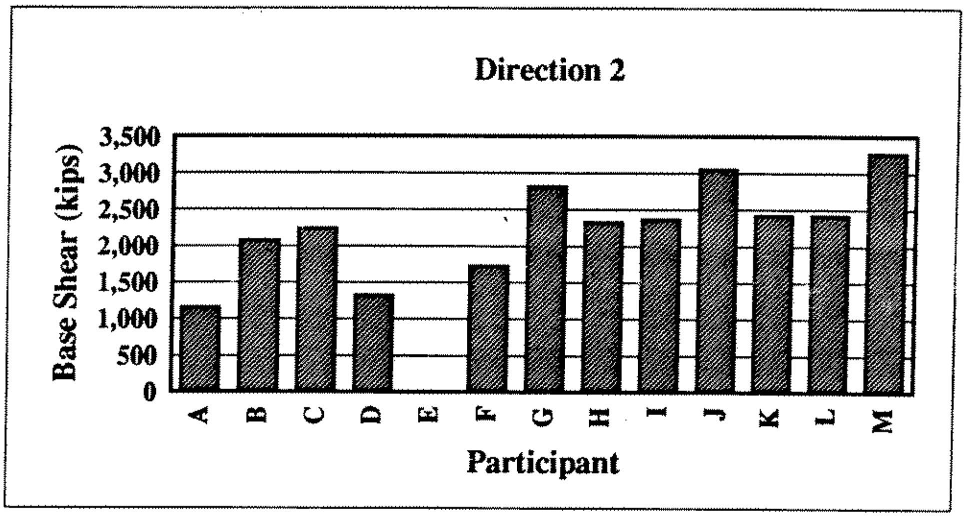


Mean	2,215
COV	0.22

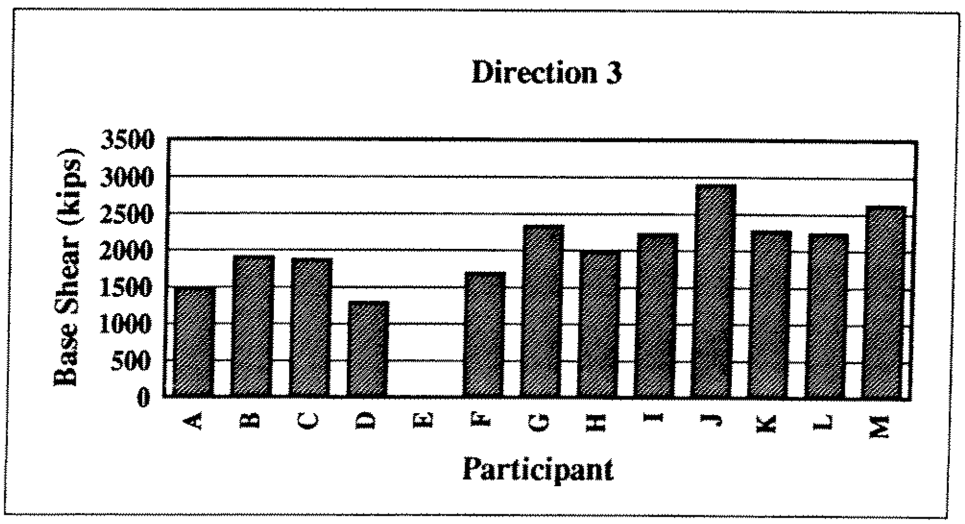
Figure 3-4: Comparison of Base Shear - Section 17 Ultimate Strength



Mean	1,725
COV	0.25

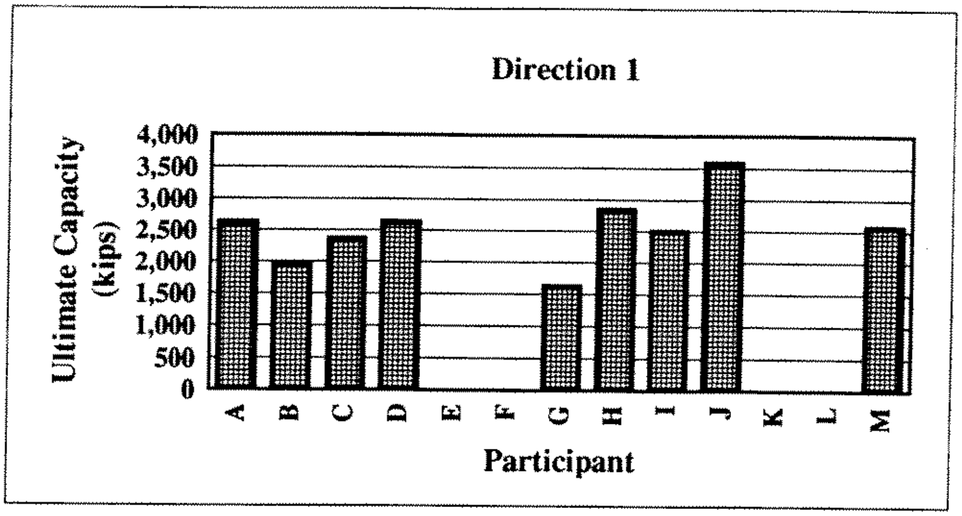


Mean	2,260
COV	0.28

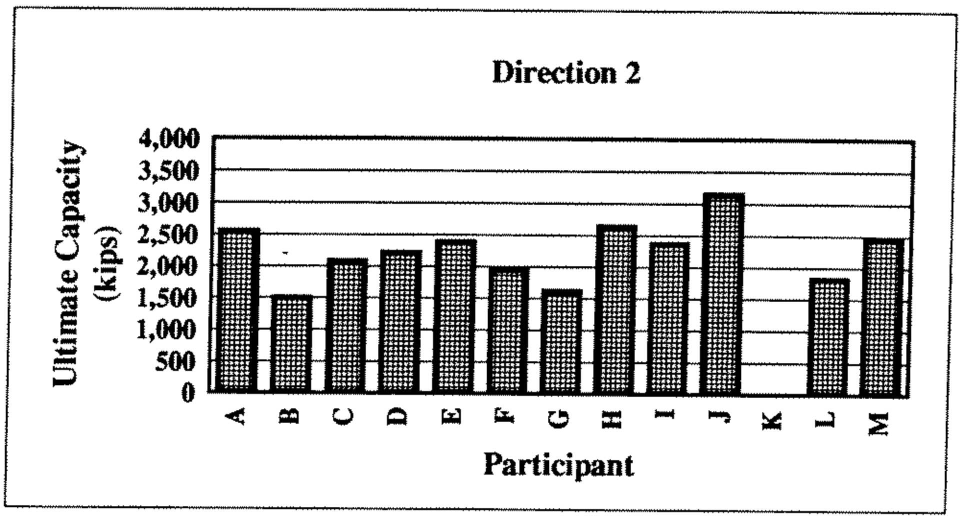


Mean	2,061
COV	0.22

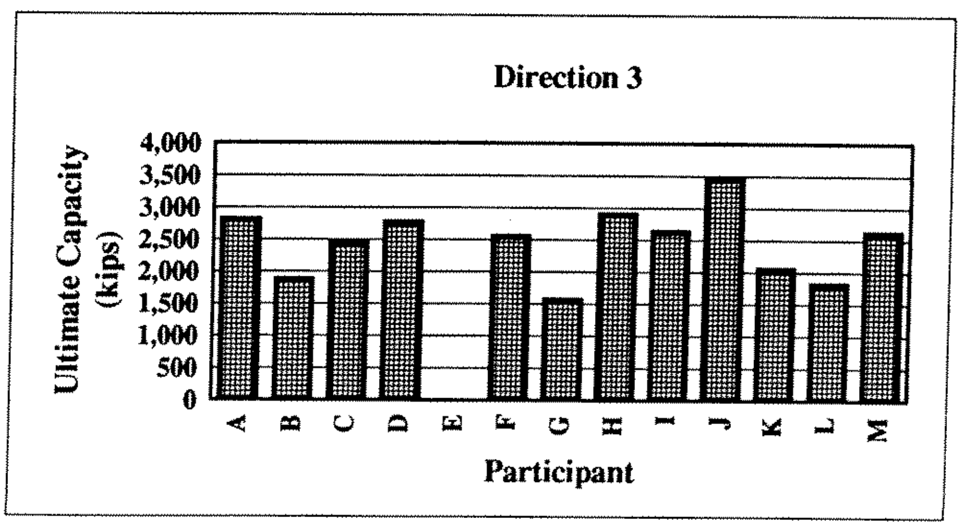
Figure 3-5: Comparison of Base Shear - RP 2A, 20th Edition



Mean	2,513
COV	0.22

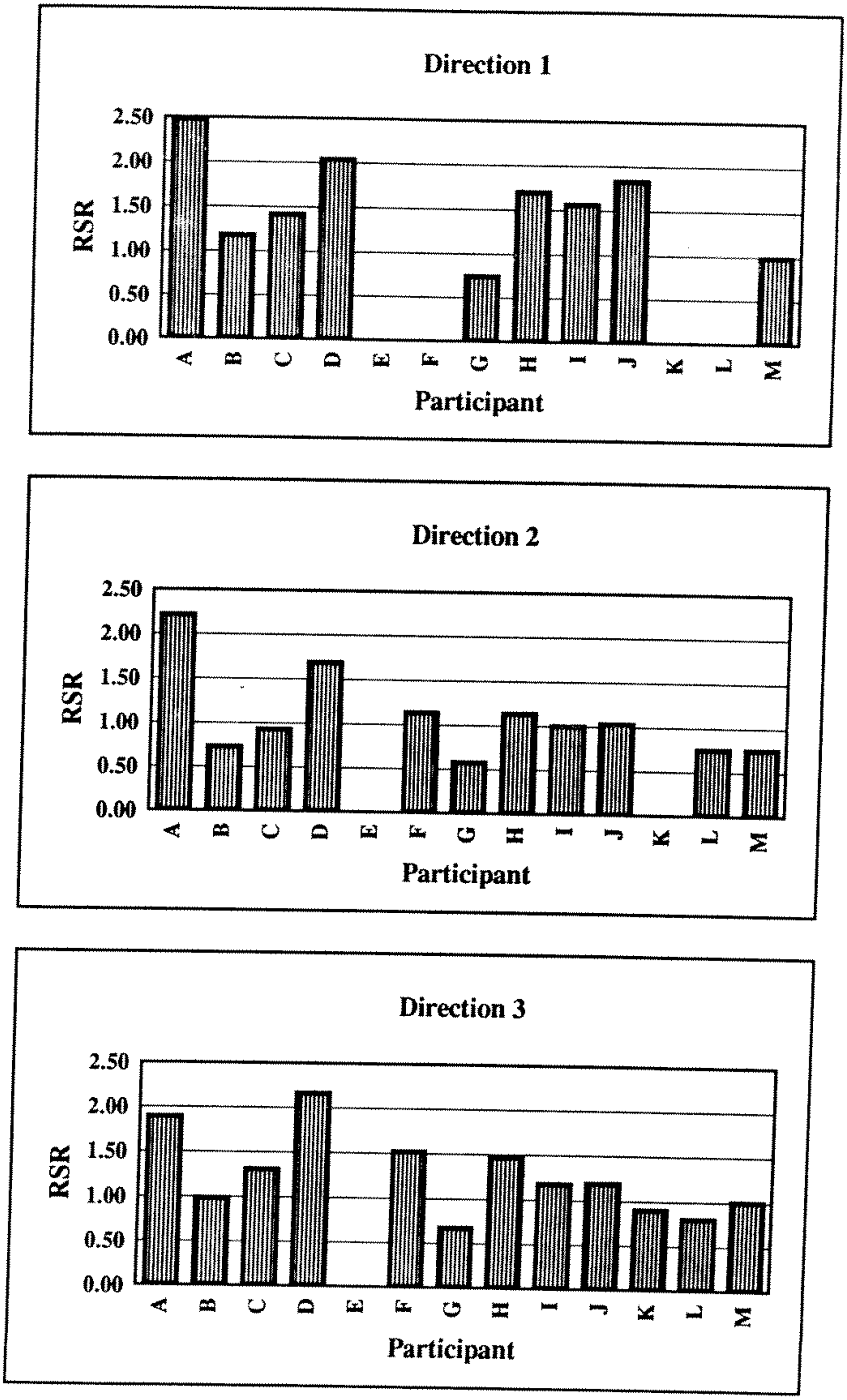


Mean	2,219
COV	0.21



Mean	2,446
COV	0.22

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



Direction 1:

Mean	1.55
COV	0.35

Direction 2:

Mean	1.08
COV	0.44

Direction 3:

Mean	1.25
COV	0.35

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

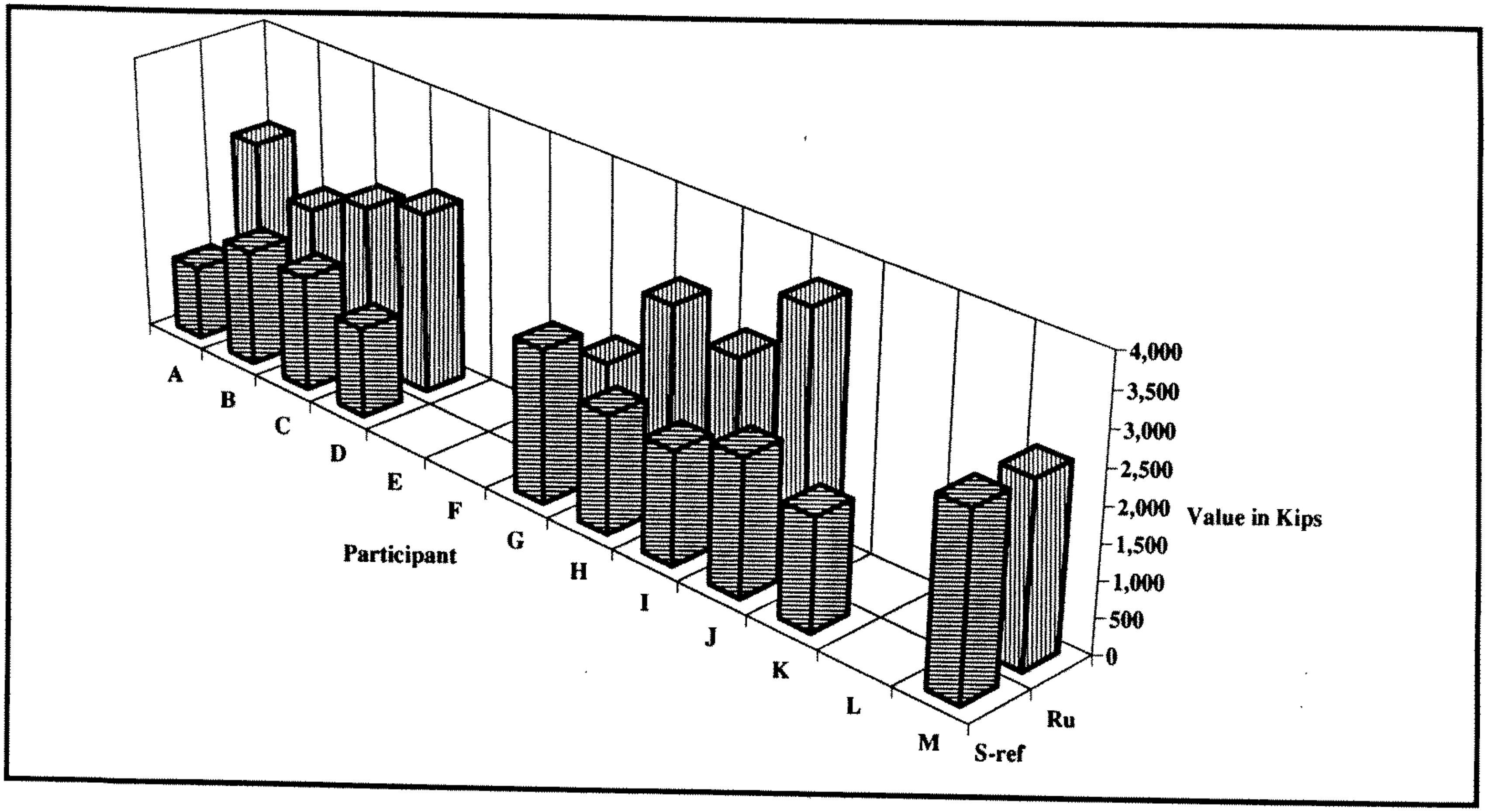


Figure 3-8(a): Variation of Reference Level Load and Ultimate Capacity - Direction 1

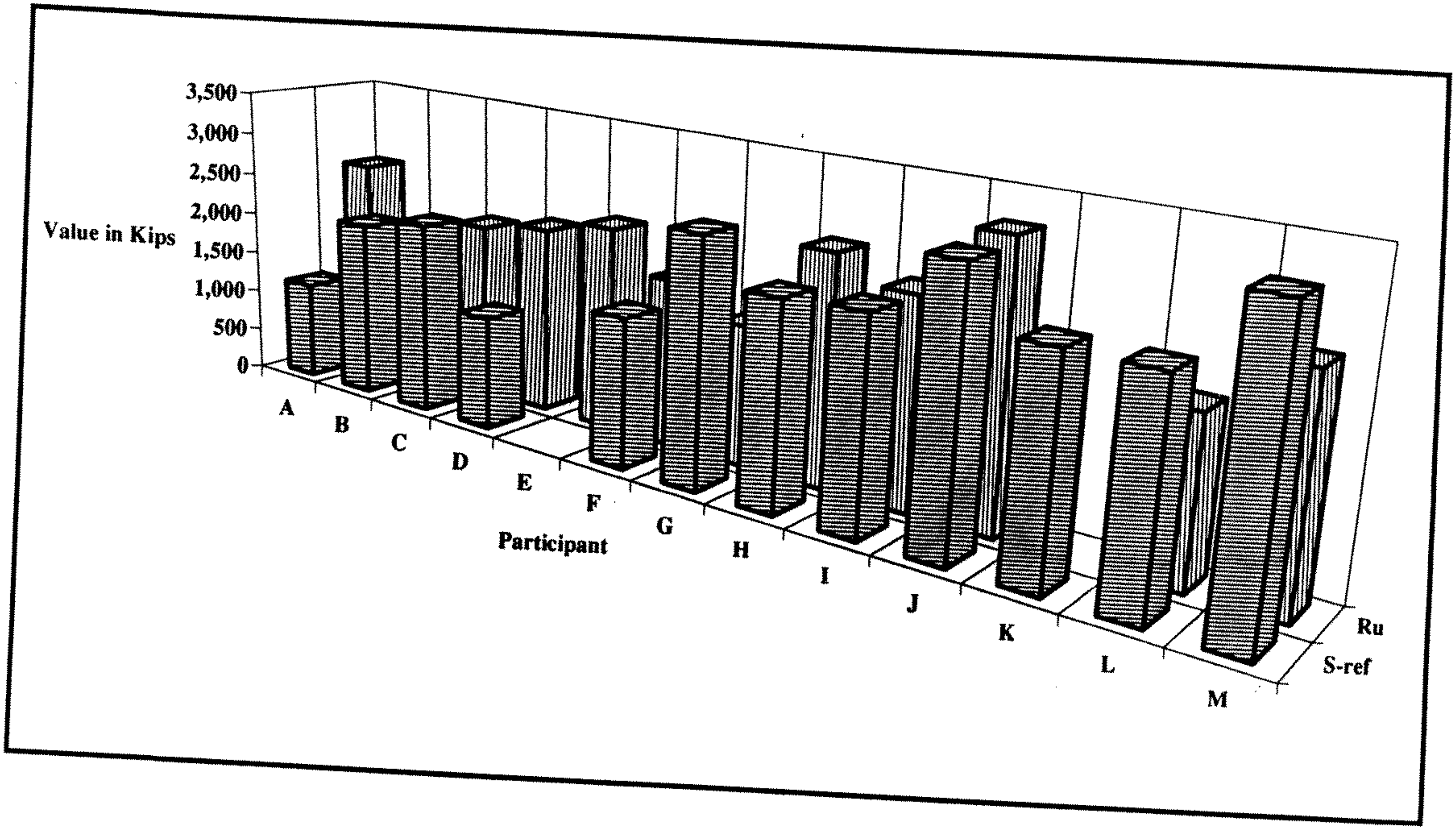


Figure 3-8(b): Variation of Reference Level Load and Ultimate Capacity - Direction 2

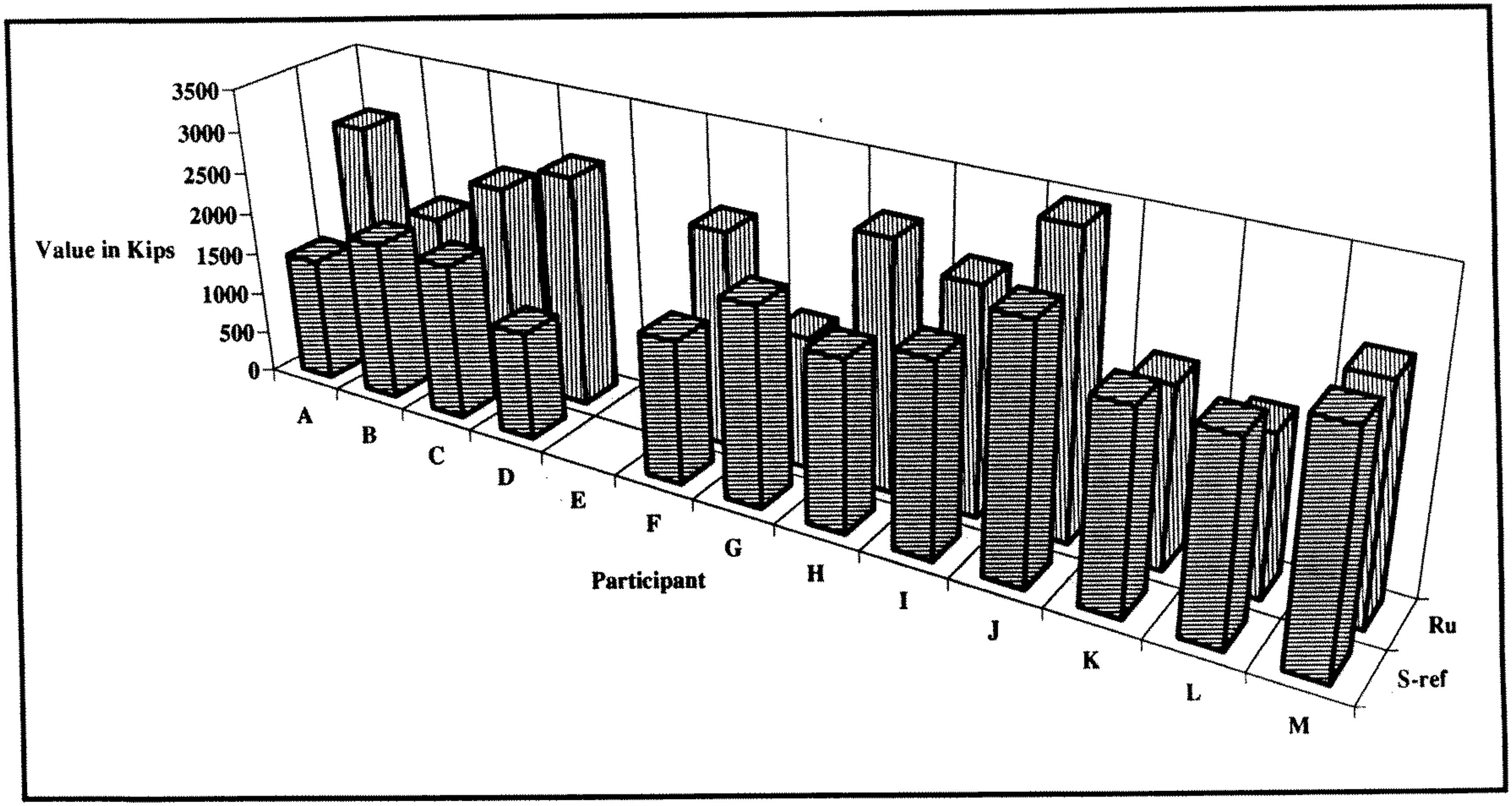


Figure 3-8(c): Variation of Reference Level Load and Ultimate Capacity - Direction 3

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

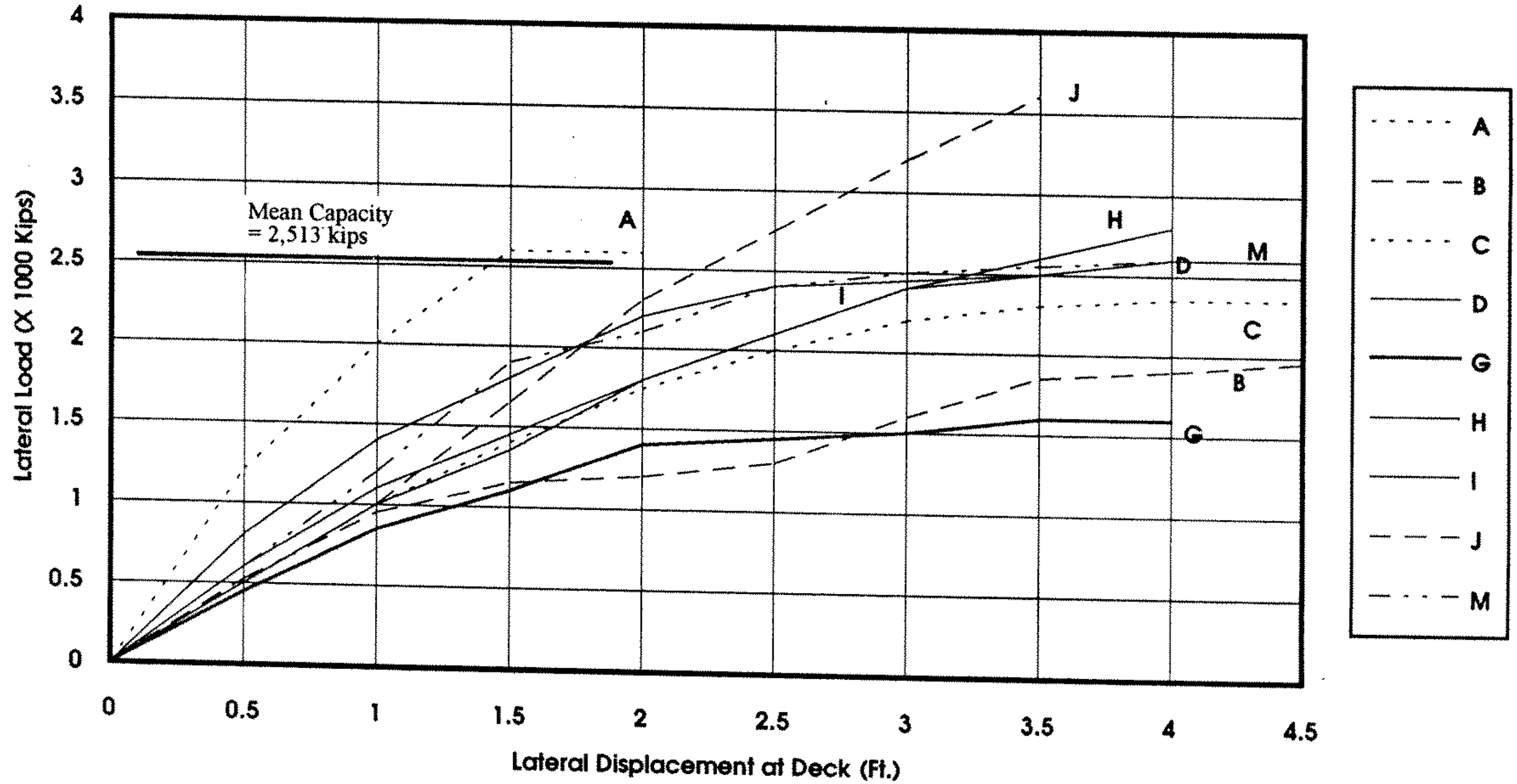


Figure 3-10 Load-Displacement Behavior - Direction 1

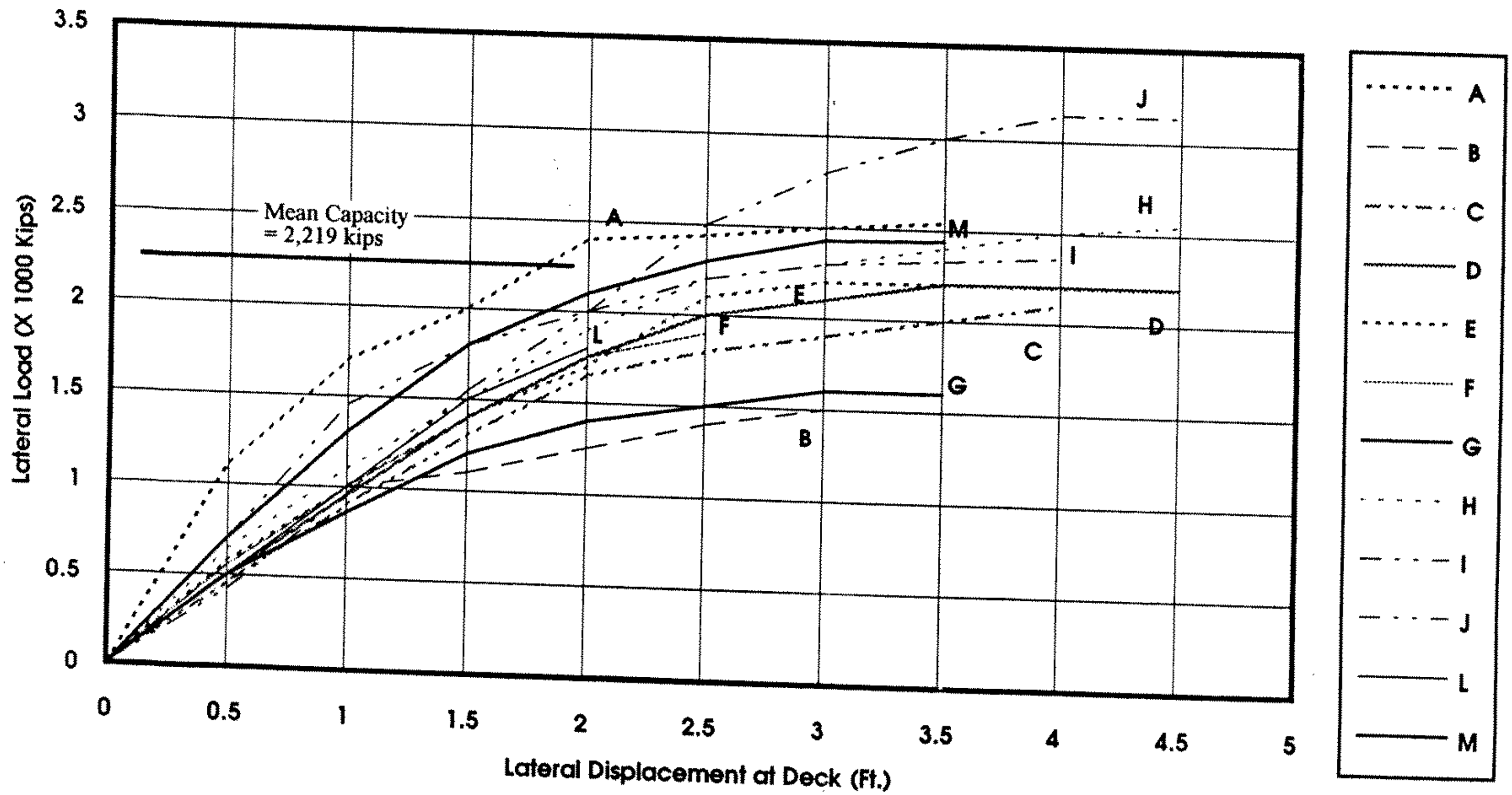


Figure 3-11 Load-Displacement Behavior - Direction 2

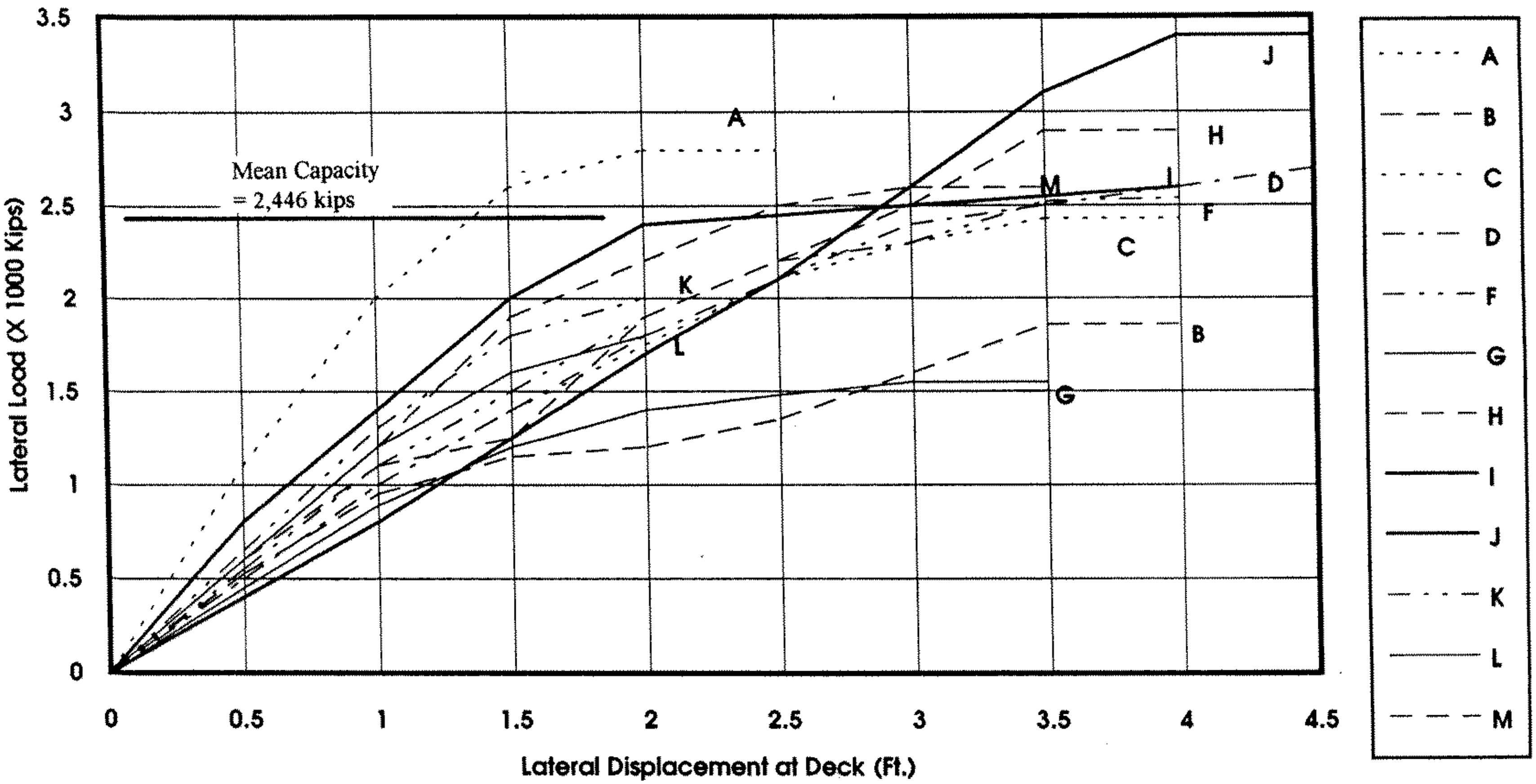
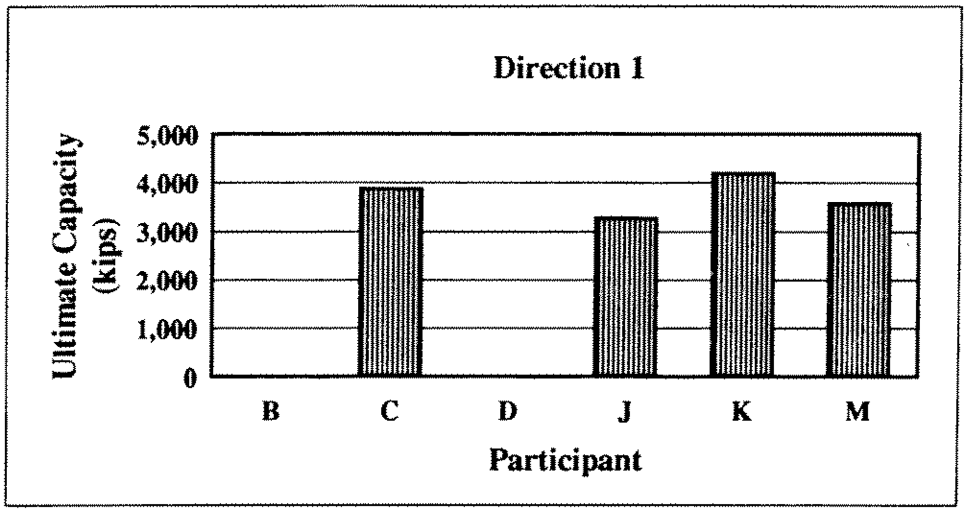
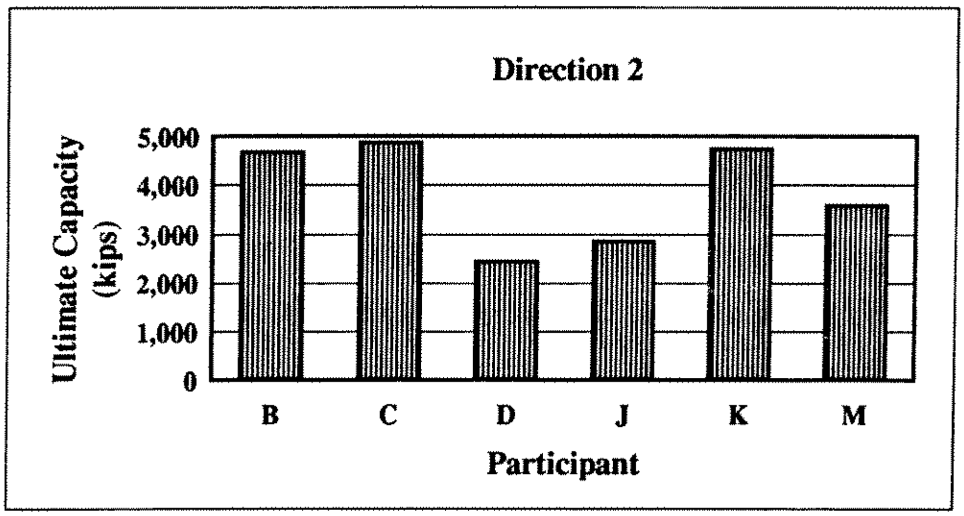


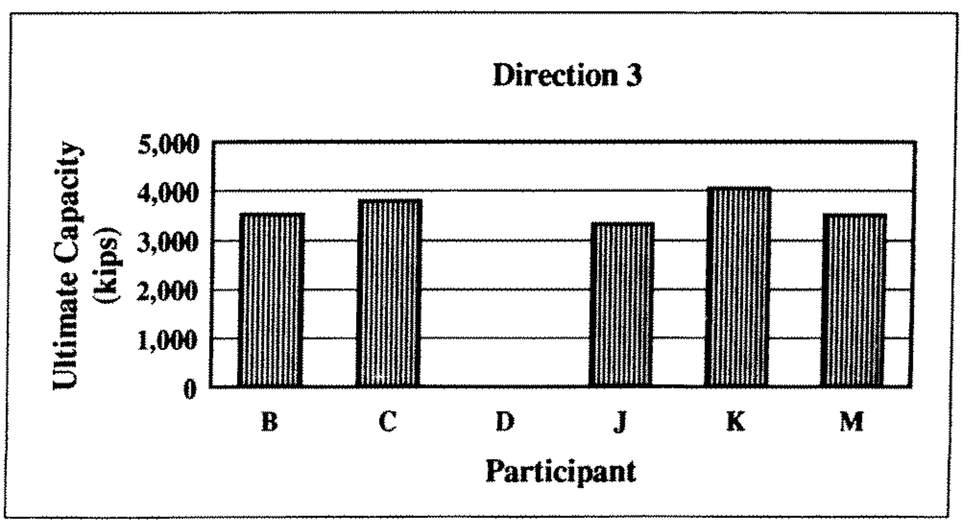
Figure 3-12 Load-Displacement Behavior - Direction 3



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

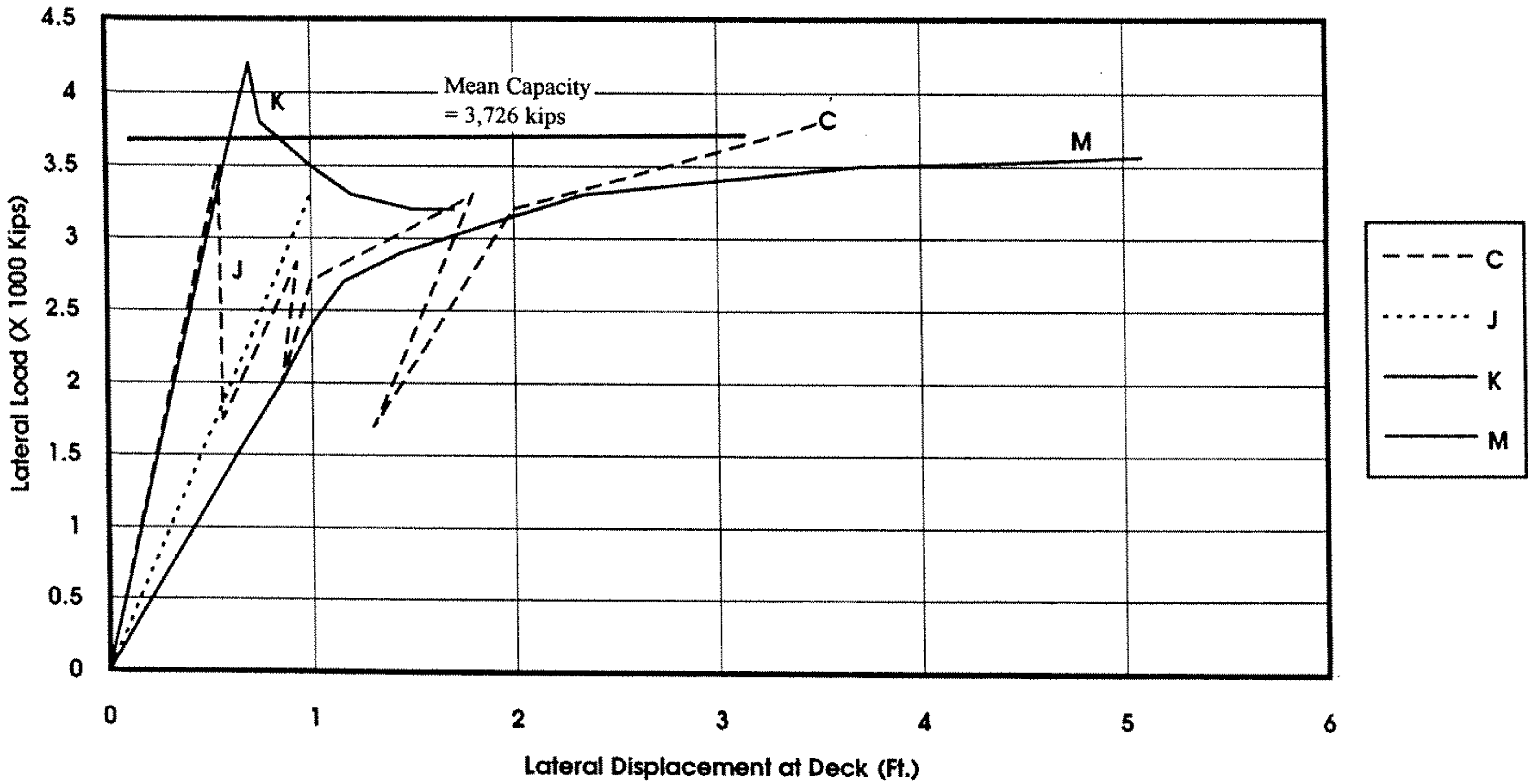


Figure 3-14 Load-Displacement Behavior - Fixed Base Case - Direction 1

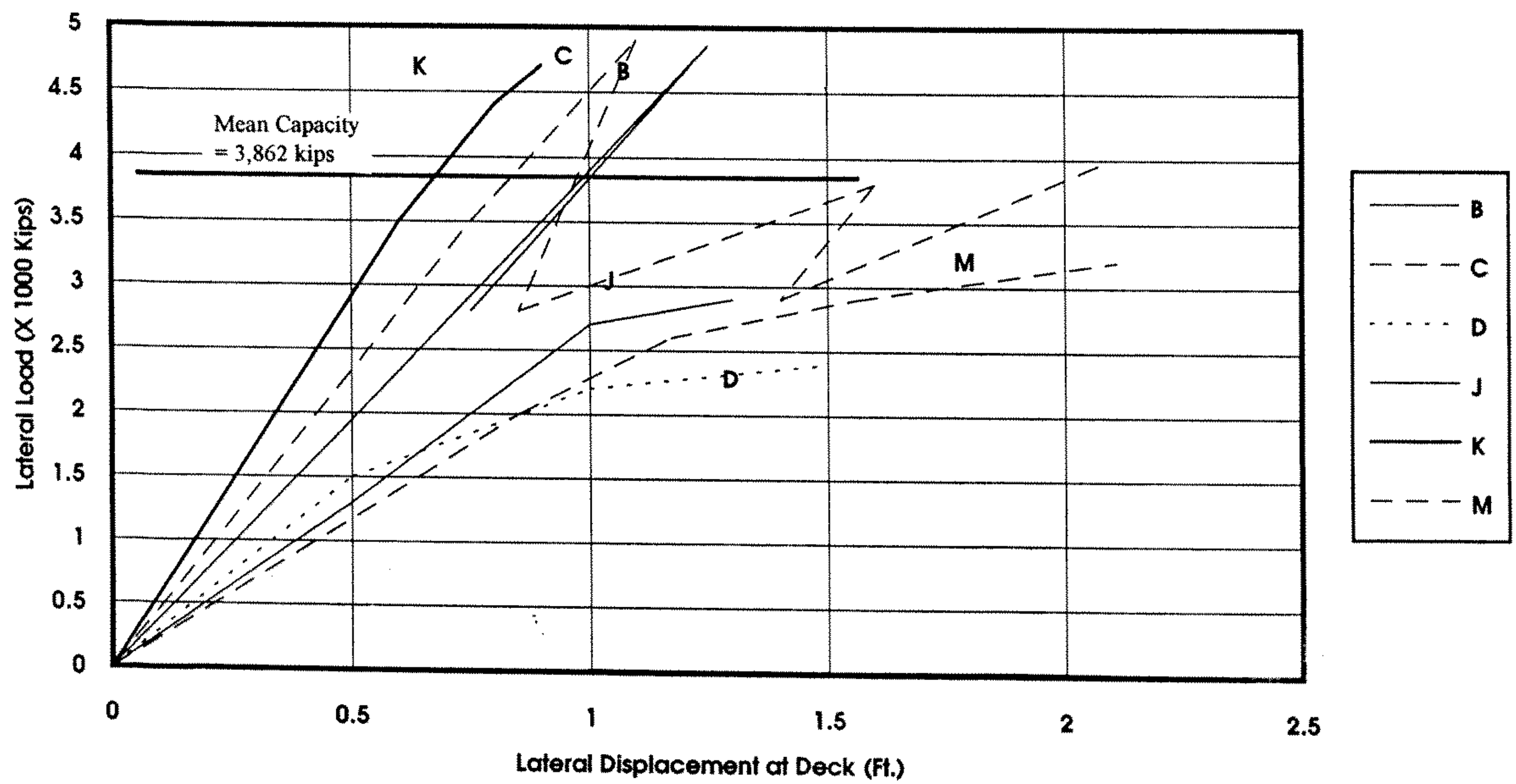


Figure 3-15 Load-Displacement Behavior - Fixed Base Case - Direction 2

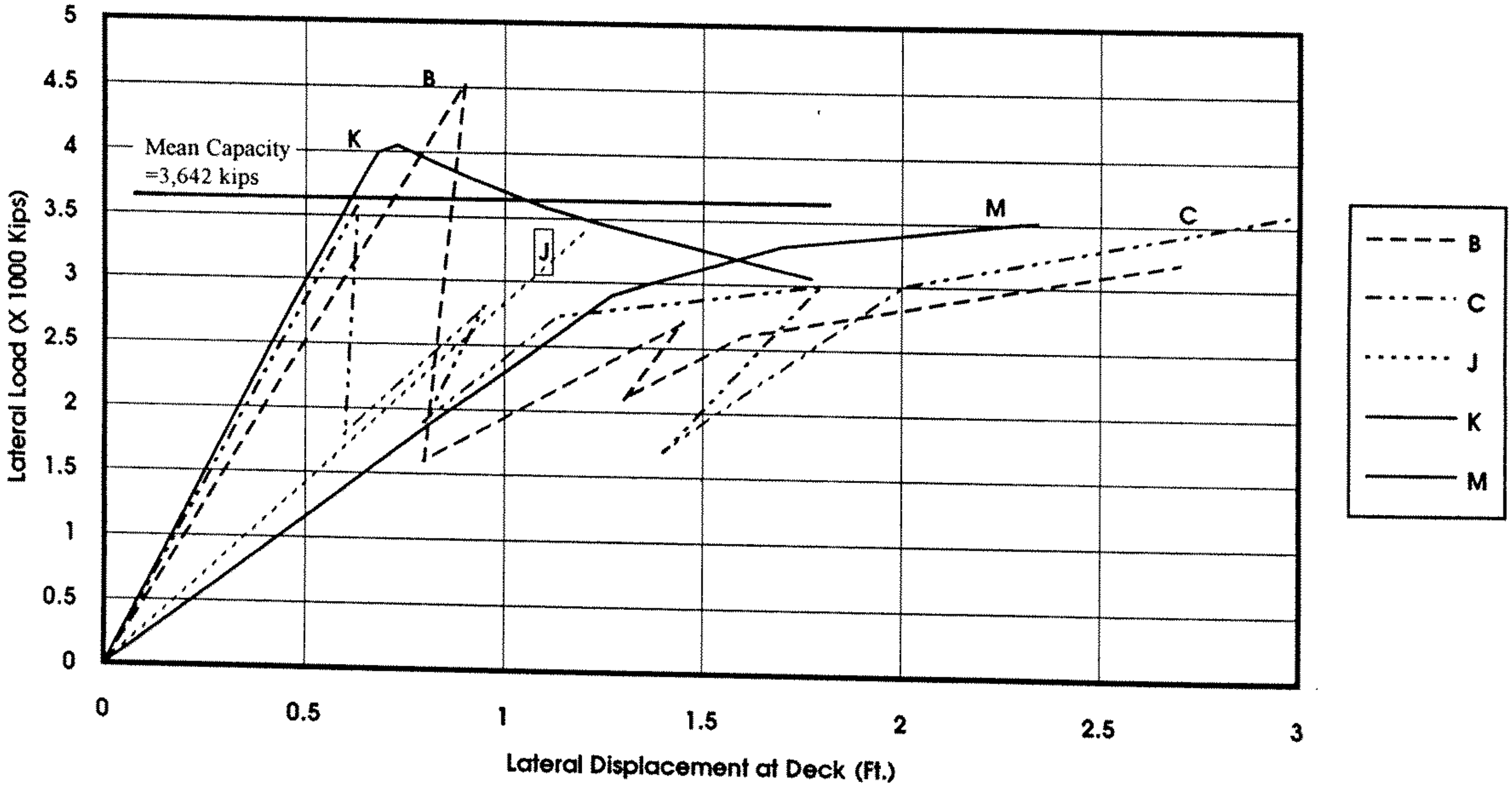


Figure 3-16 Load-Displacement Behavior - Fixed Base Case - Direction 3

	Range	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)**

	Range	Mean	Cov (%)	Per API TG
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)**

	Range	Mean	Cov (%)
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)**

	Range	Mean	Cov (%)
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)

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

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

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

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 3-8: 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	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.

- <u>Section 17.7.3c and C17.7.3c – Global Inelastic Analysis</u>

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.

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

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.

Parameter	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)**

Parameter	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)**

Parameter	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)**

Parameter	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

.046\TRIALS\FIG-A-1

	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

	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: 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)	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 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 1st Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at 1st Member with NL Event (kips)	Ultimate Capacity, Ru (kips)	RSR = Ru/S-20	Failure Mode/ Mechanism	Base Shear, S-20 (kips)	Load at 1st 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	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	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**

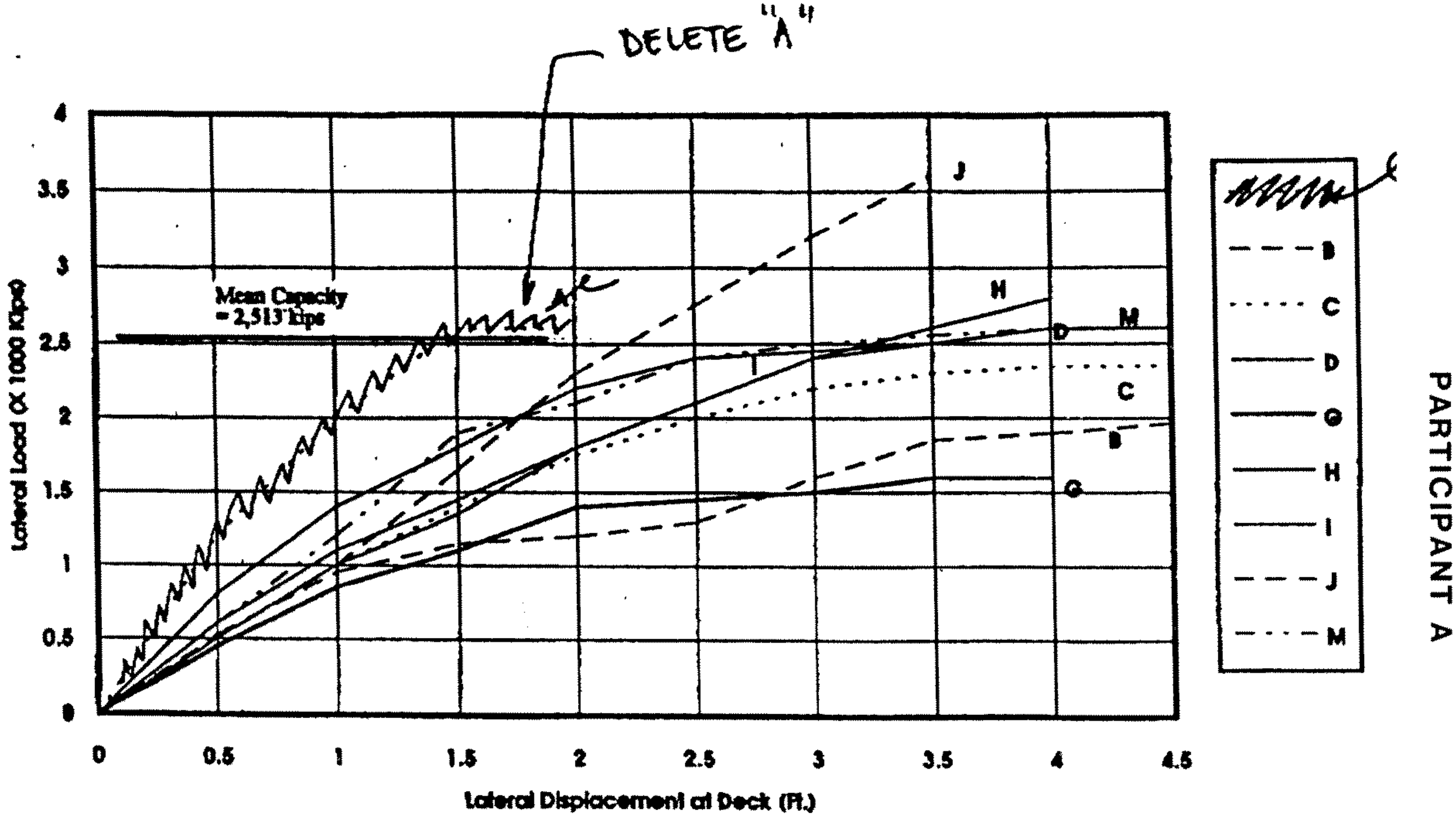


Figure 3-10 Load-Displacement Behavior - Direction 1

PARTICIPANT A

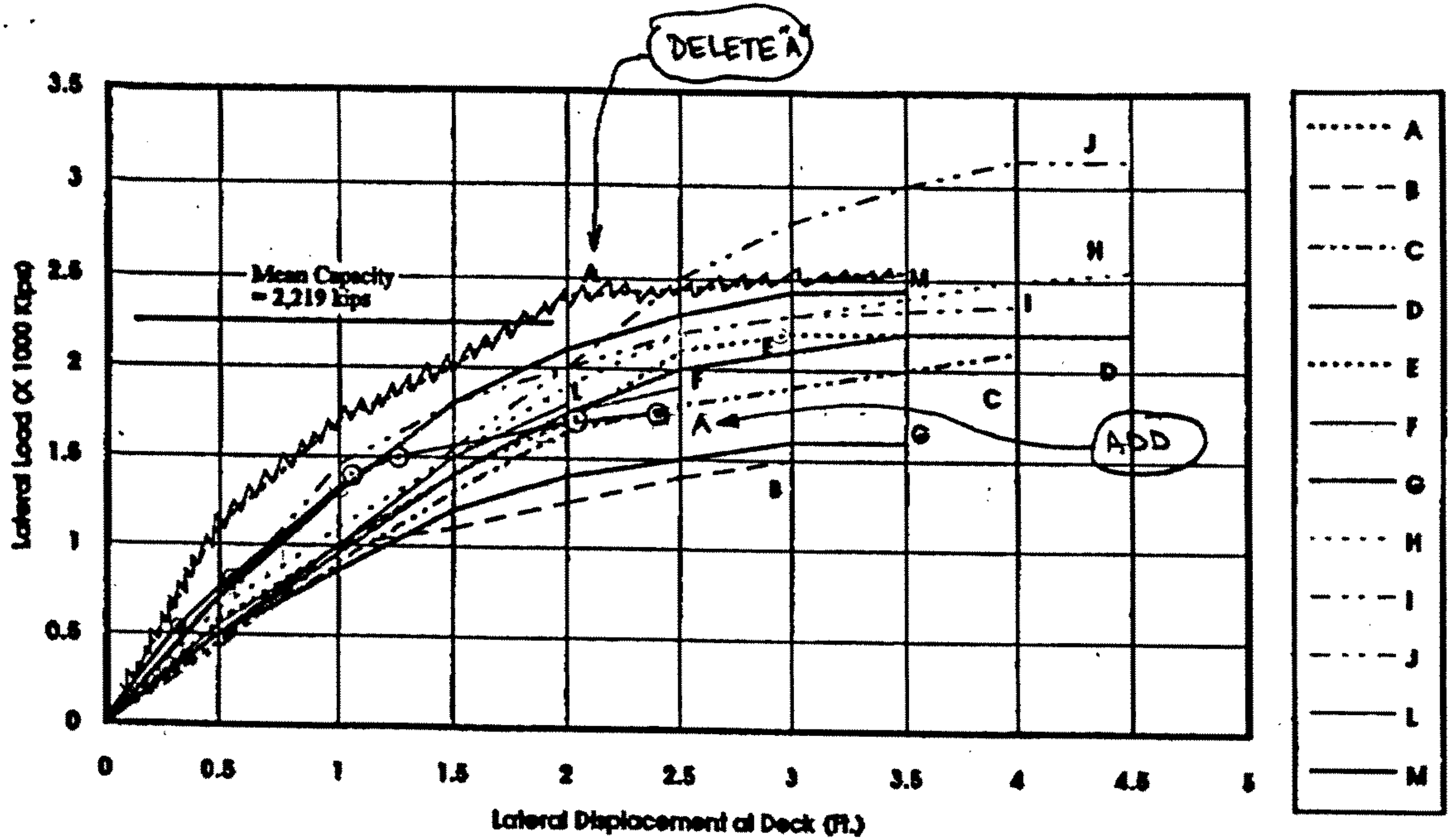


Figure 3-11 Load-Displacement Behavior - Direction 2

PARTICIPANT A

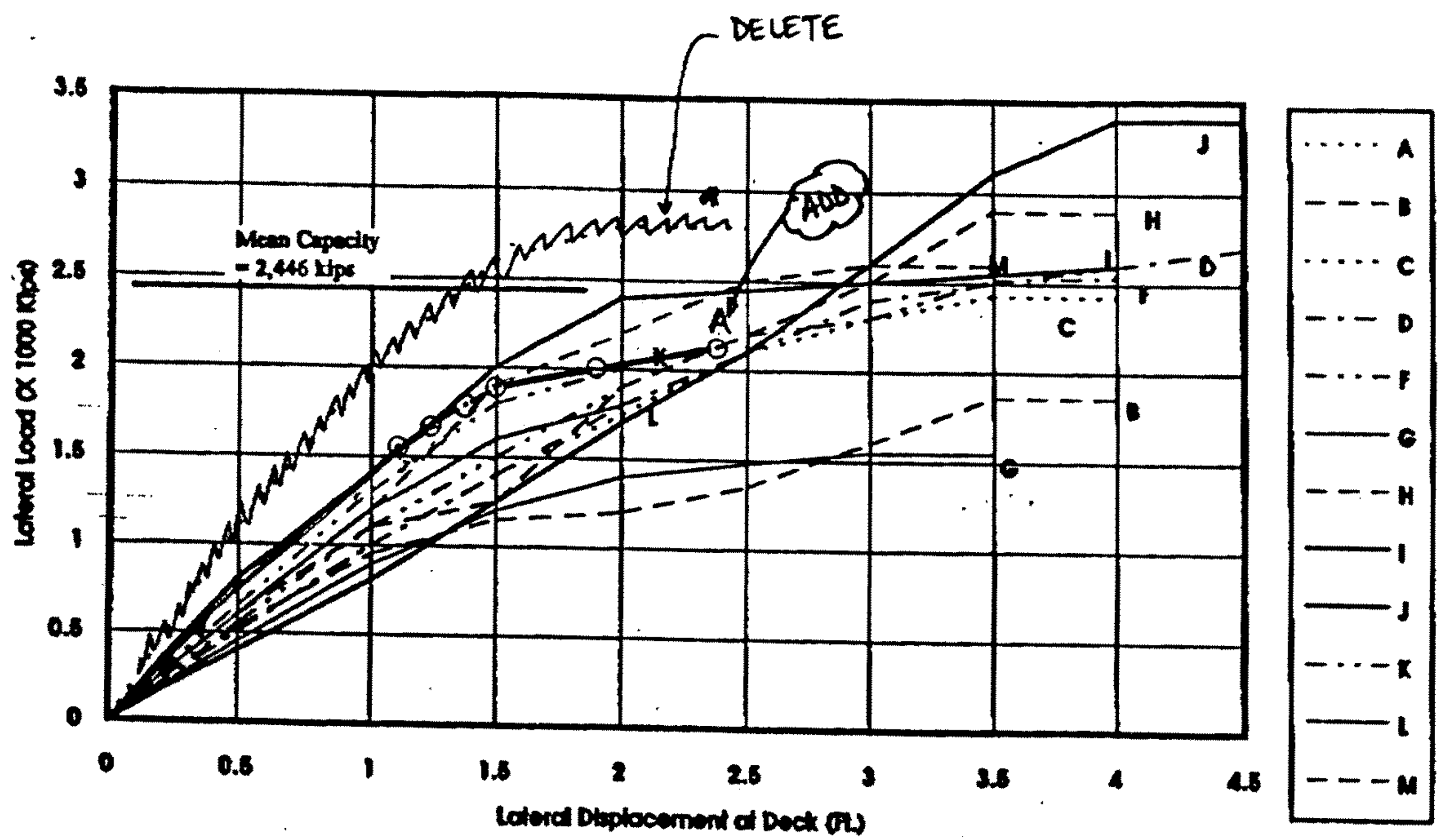


Figure 3-12 Load-Displacement Behavior - Direction 3

PARTICIPANT K

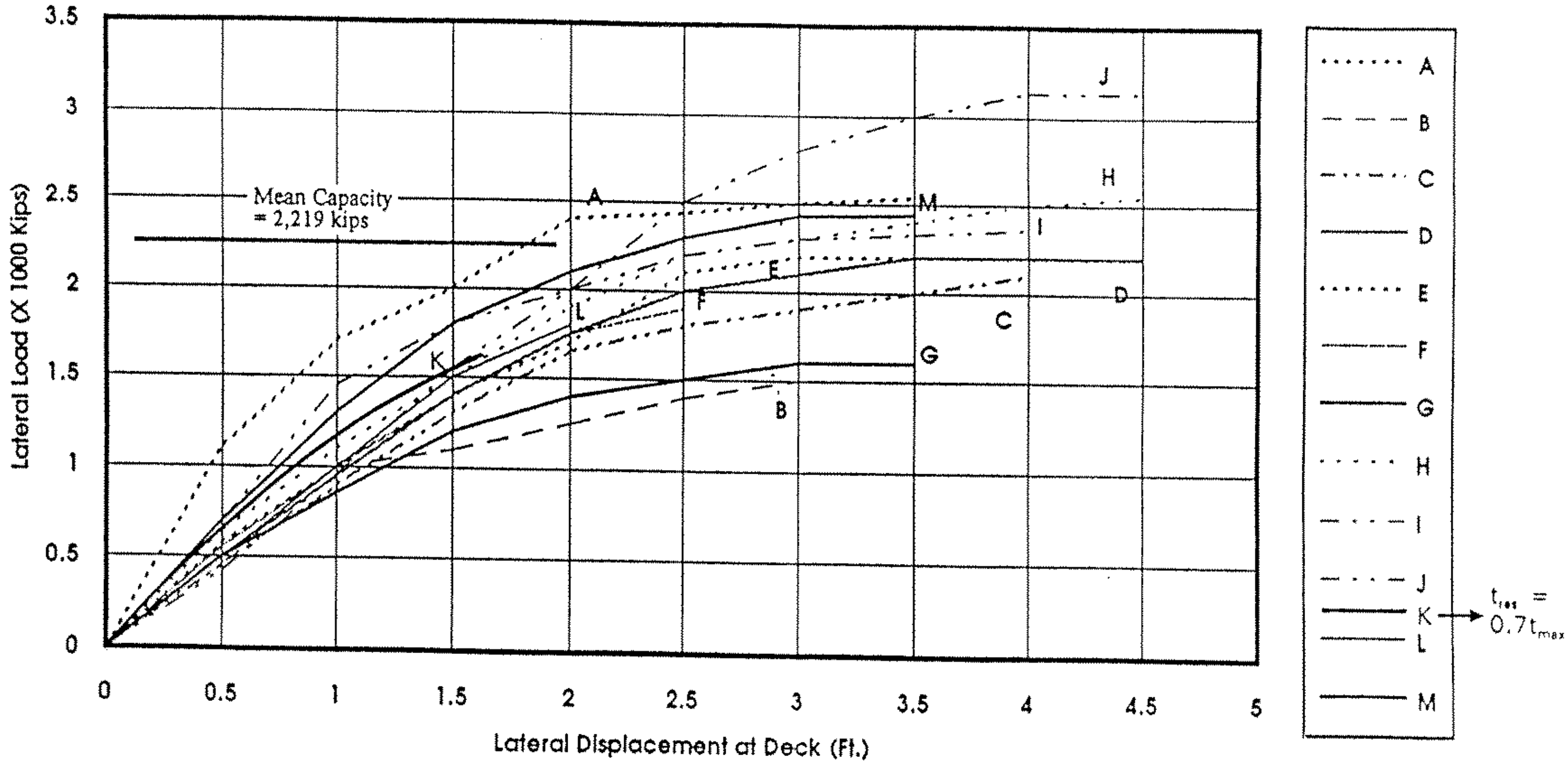


Figure 3-11 Load-Displacement Behavior - Direction 2

PARTICIPANT K

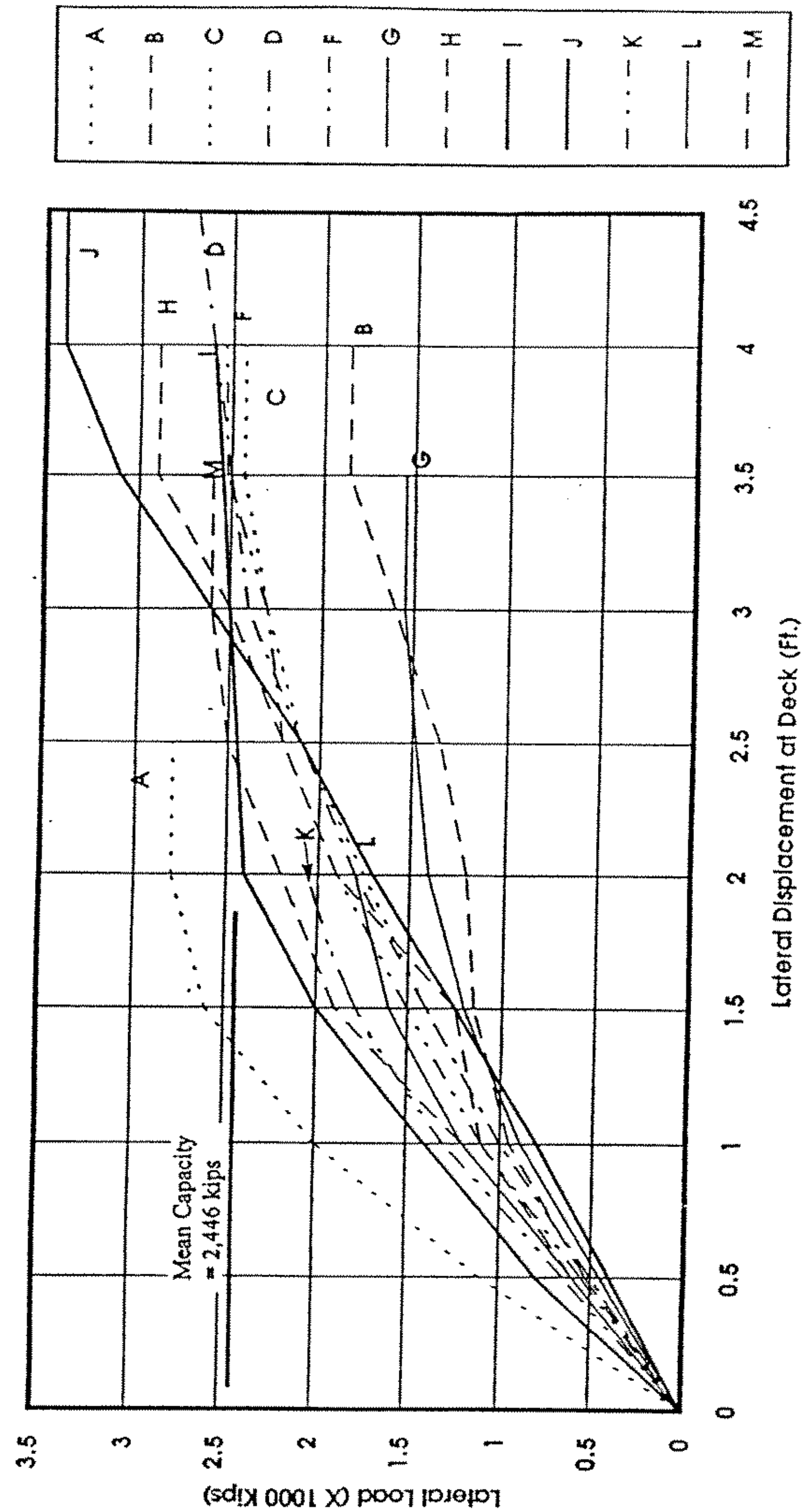


Figure 3-12 Load-Displacement Behavior - Direction 3

PARTICIPANT K

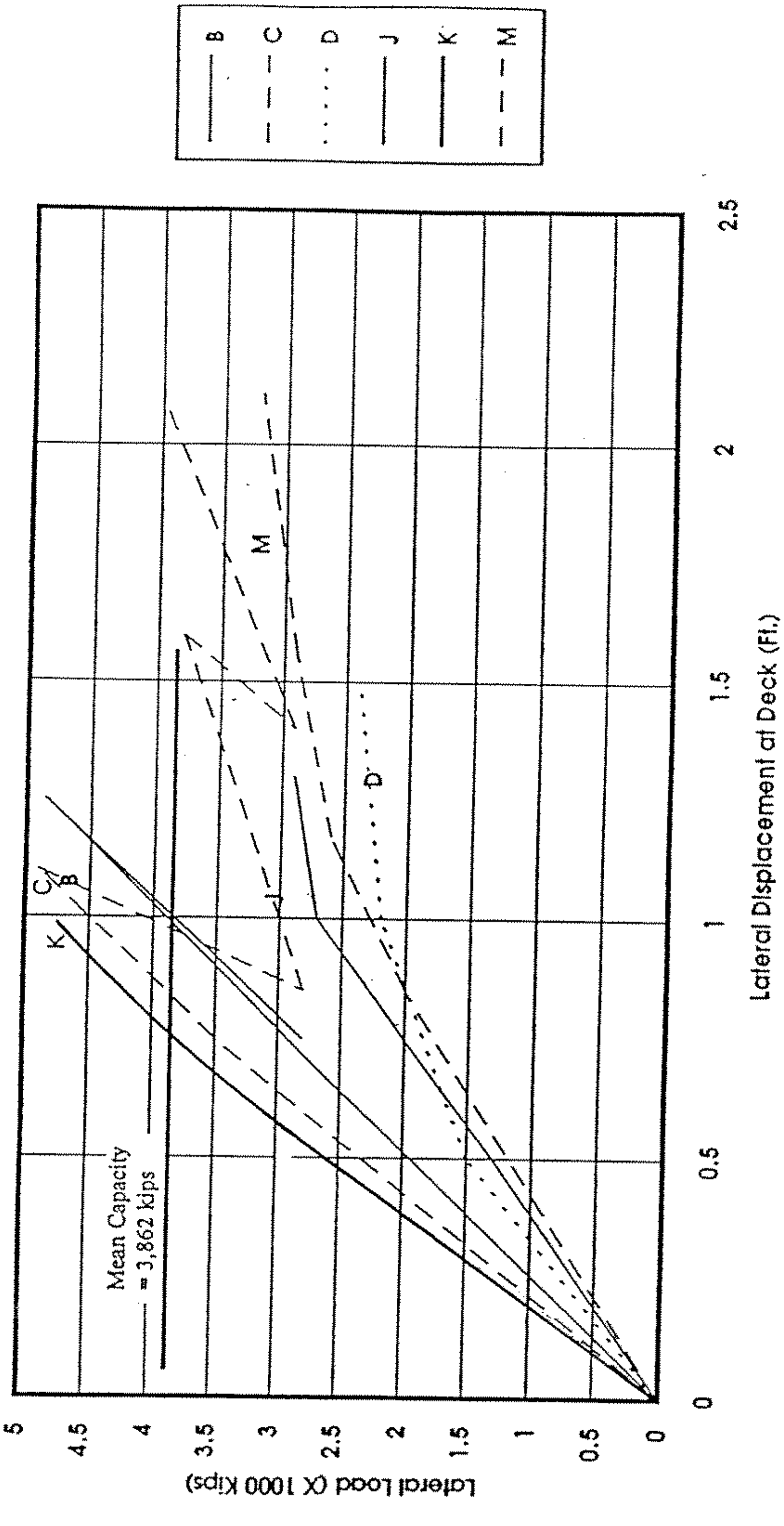


Figure 3-15 Load-Displacement Behavior - Fixed Base Case - Direction 2

PARTICIPANT K

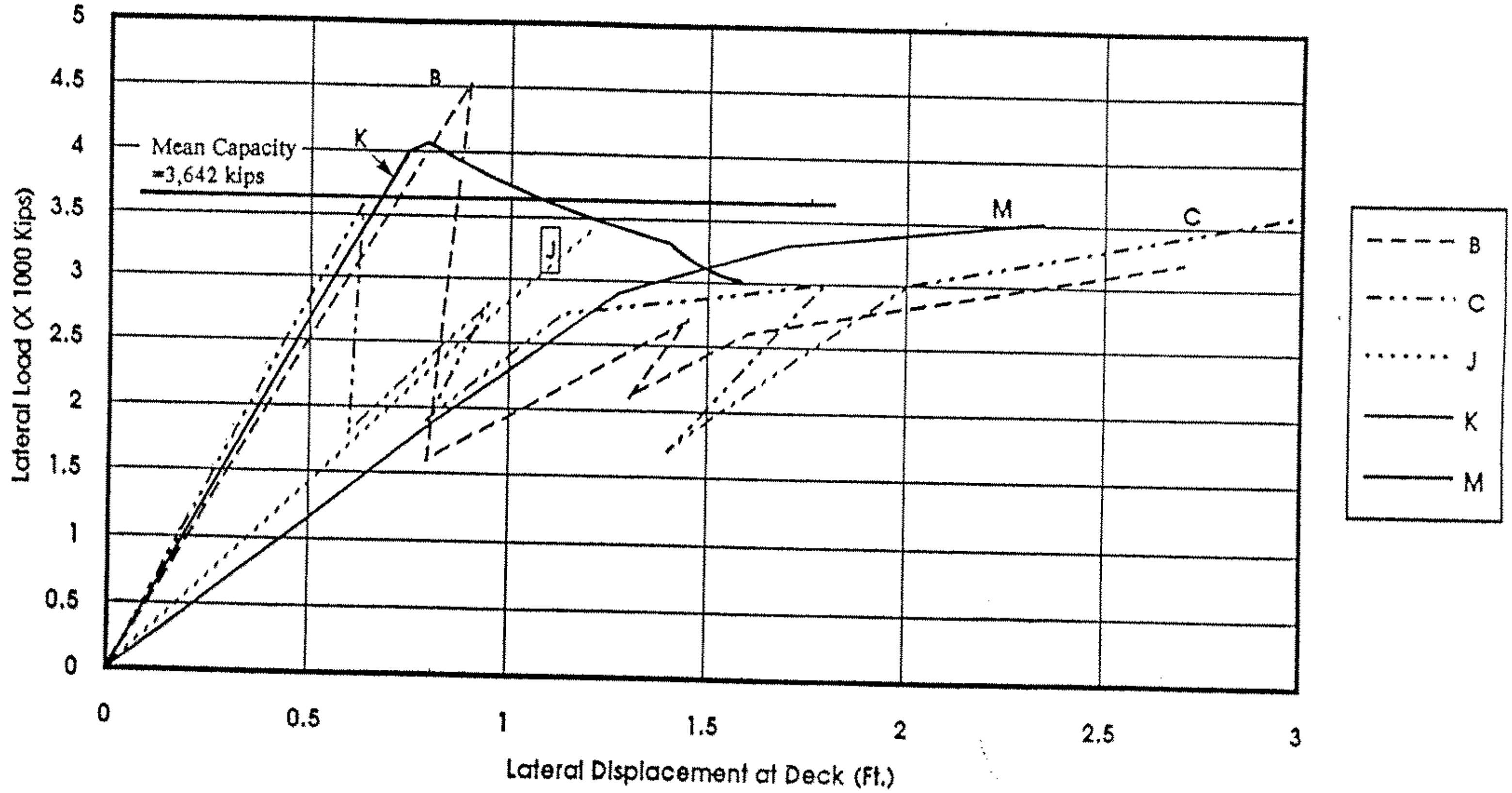


Figure 3-16 Load-Displacement Behavior - Fixed Base Case - Direction 3

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.

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

4. Sufficient static push-over analysis should be performed to determine the <u>MINIMUM</u> 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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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
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 ≥ 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

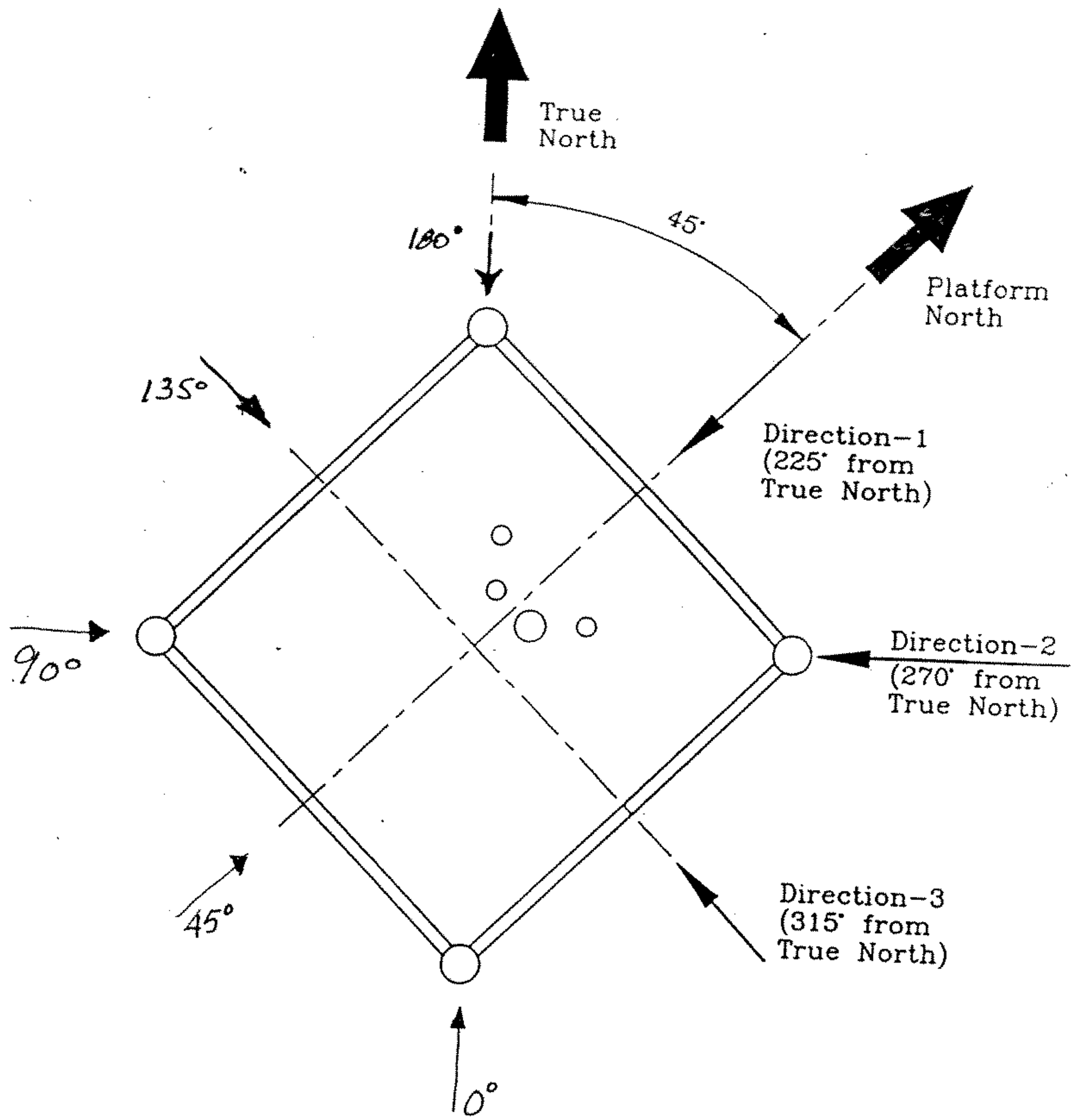
Marine Growth	Thickness = 1.5" (from + 1.0 ft to -150.0 ft)
Wave Kin. Factor	0.88
Current Blockage Factor	0.80 for end-on and tranverse directions 0.85 for diagonal directions
Current Profile	Uniform over the water column
Force Coeff.	Use default values Default values are: Cd(smooth) = 0.65, Cd(rough) = 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



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

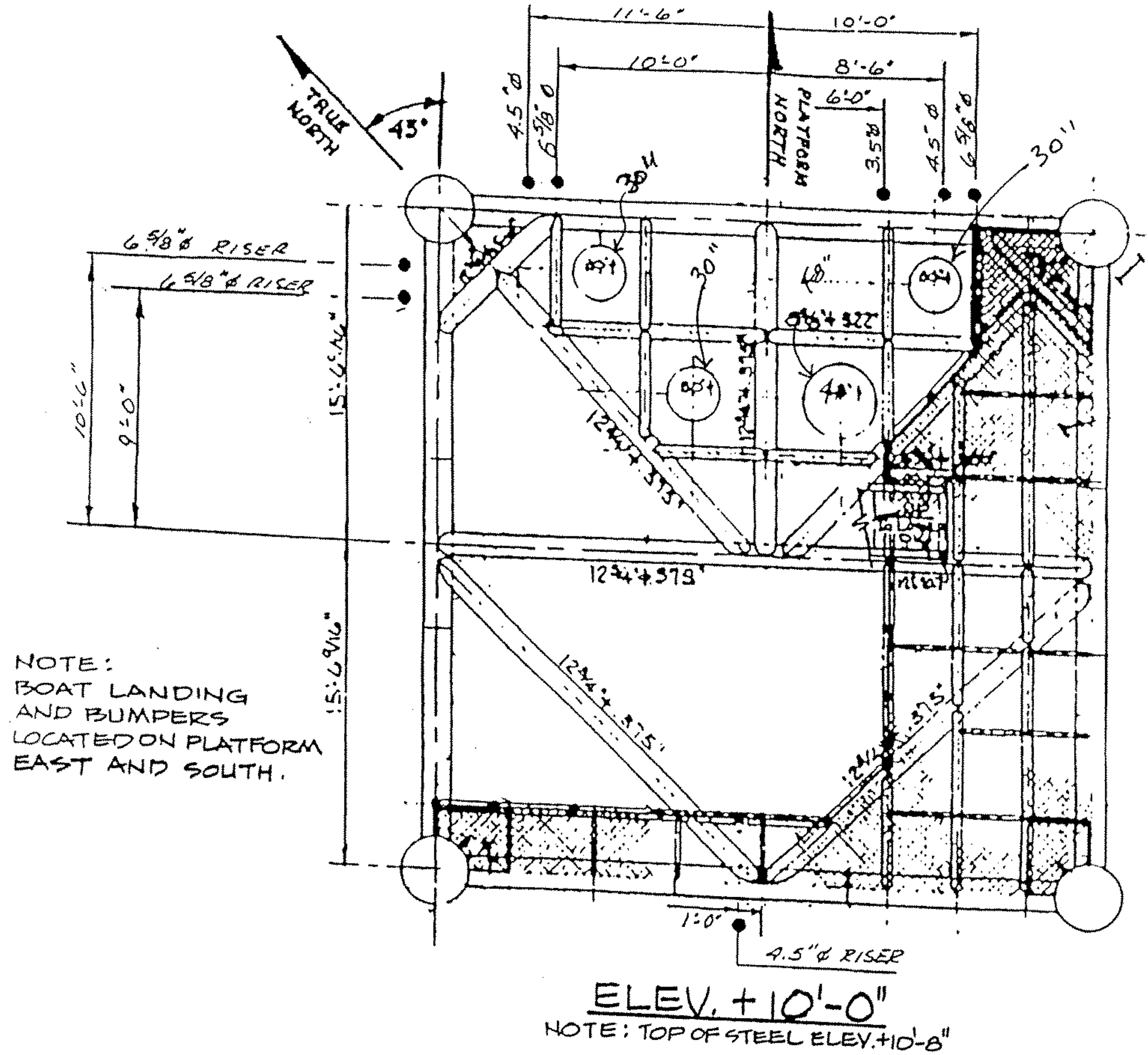


Figure 2[illegible] Key Plan - Benchmark Platform

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.

'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.

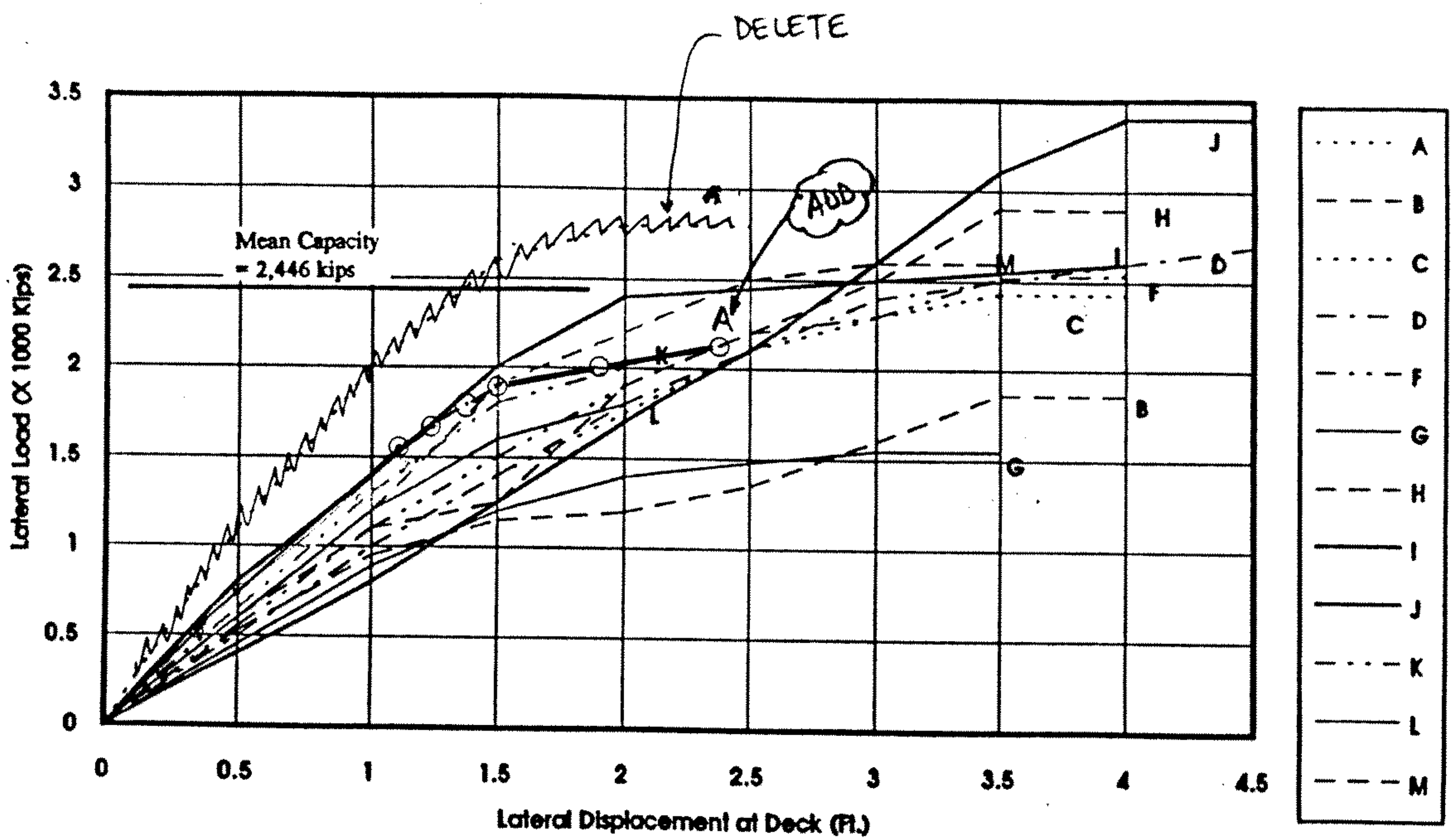


Figure 3-12 Load-Displacement Behavior - Direction 3

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

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

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?

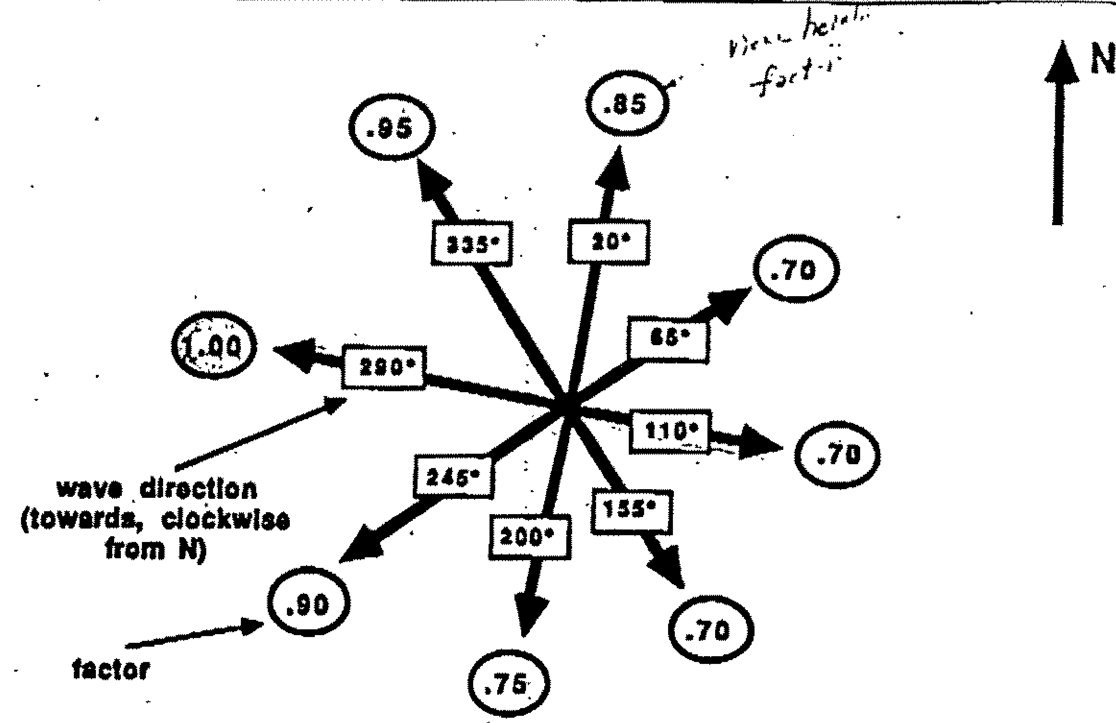


FIG. 2.3.4-4
GUIDELINE DESIGN WAVE DIRECTIONS AND FACTORS TO APPLY TO THE OMNIDIRECTIONAL WAVE HEIGHTS (FIG. 2.3.4-3), GULF OF MEXICO, NORTH OF 27° N AND WEST OF 86° W

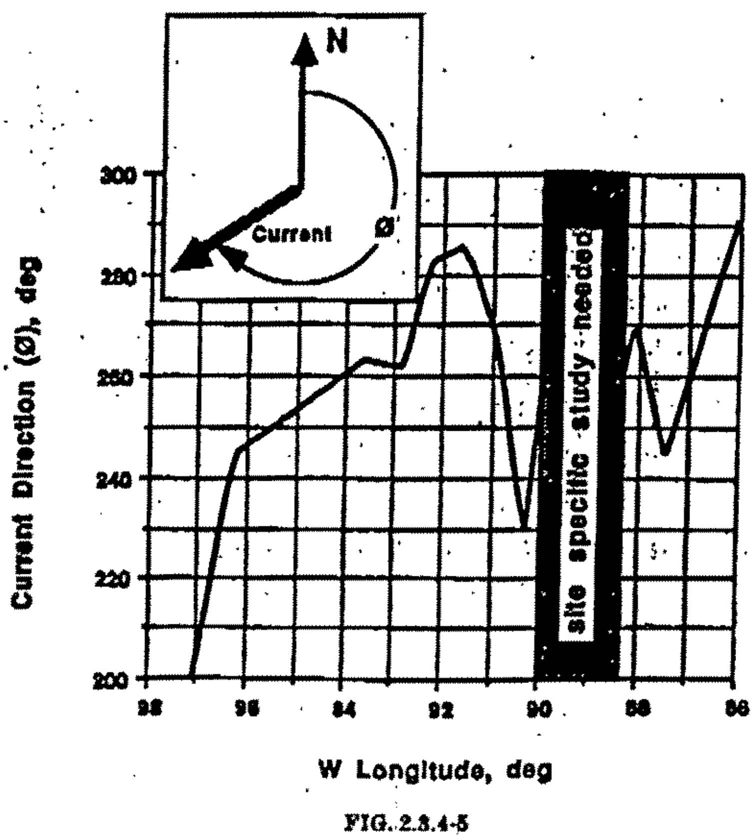


FIG. 2.3.4-5
GUIDELINE DESIGN CURRENT DIRECTION (TOWARDS) WITH RESPECT TO NORTH IN SHALLOW WATER (DEPTH ≤ 150 FT), GULF OF MEXICO, NORTH OF 27° N AND WEST OF 86° W

PARTICIPANT I

ADDITIONAL INFORMATION ON BENCHMARK STUDY

1. Structural Modeling

The following simplifications were made in the structural model :

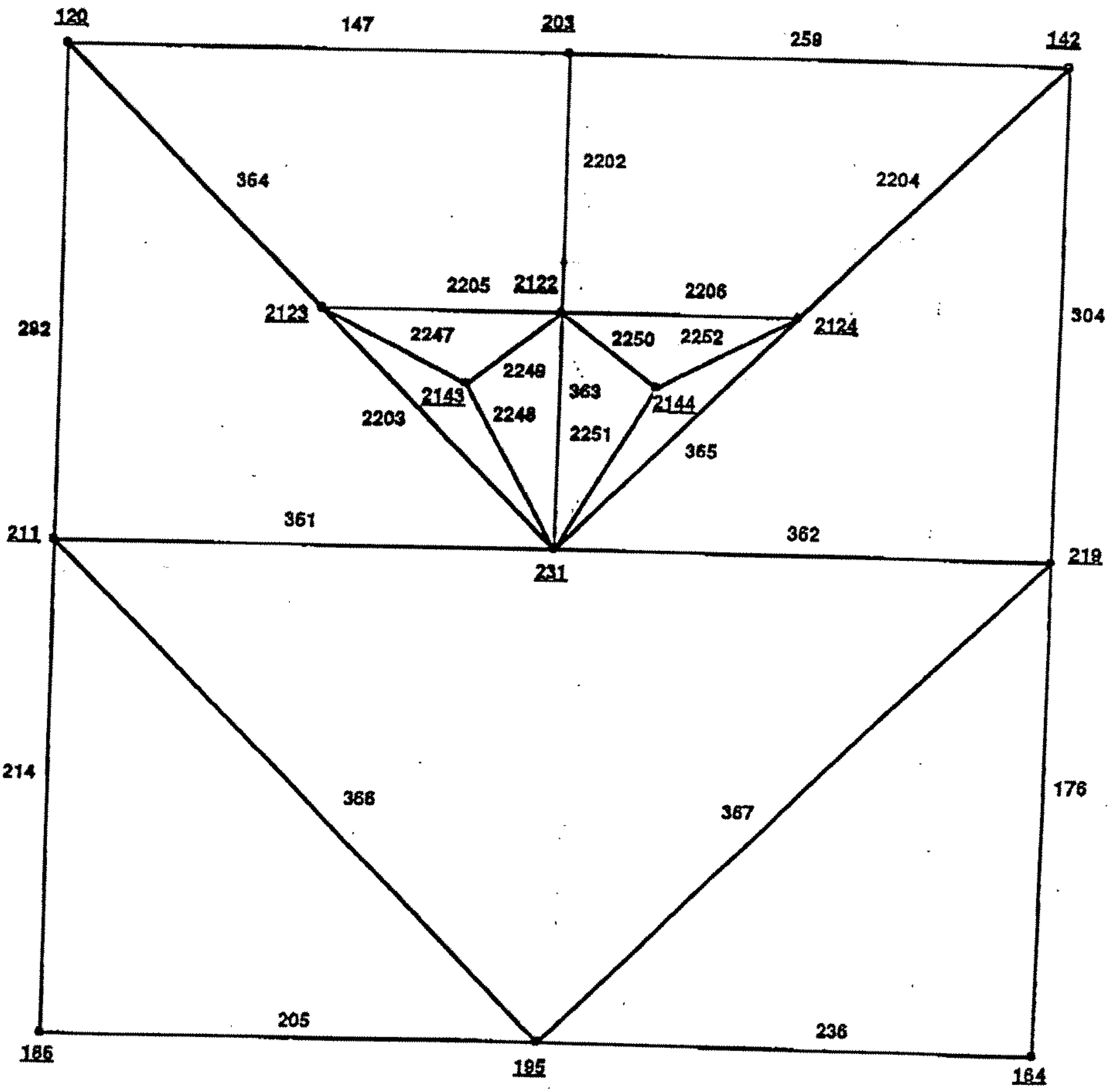
- Conductor Support at the Mudline - The conductor support is a hinge support at the mudline (Constraints in X,Y and Z directions - displacements only).
- Conductor Framing - The conductor framing is simplified. Did not use the actual framing as provided by PMB
- Conductor Modeling - Simplified. Use 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 "

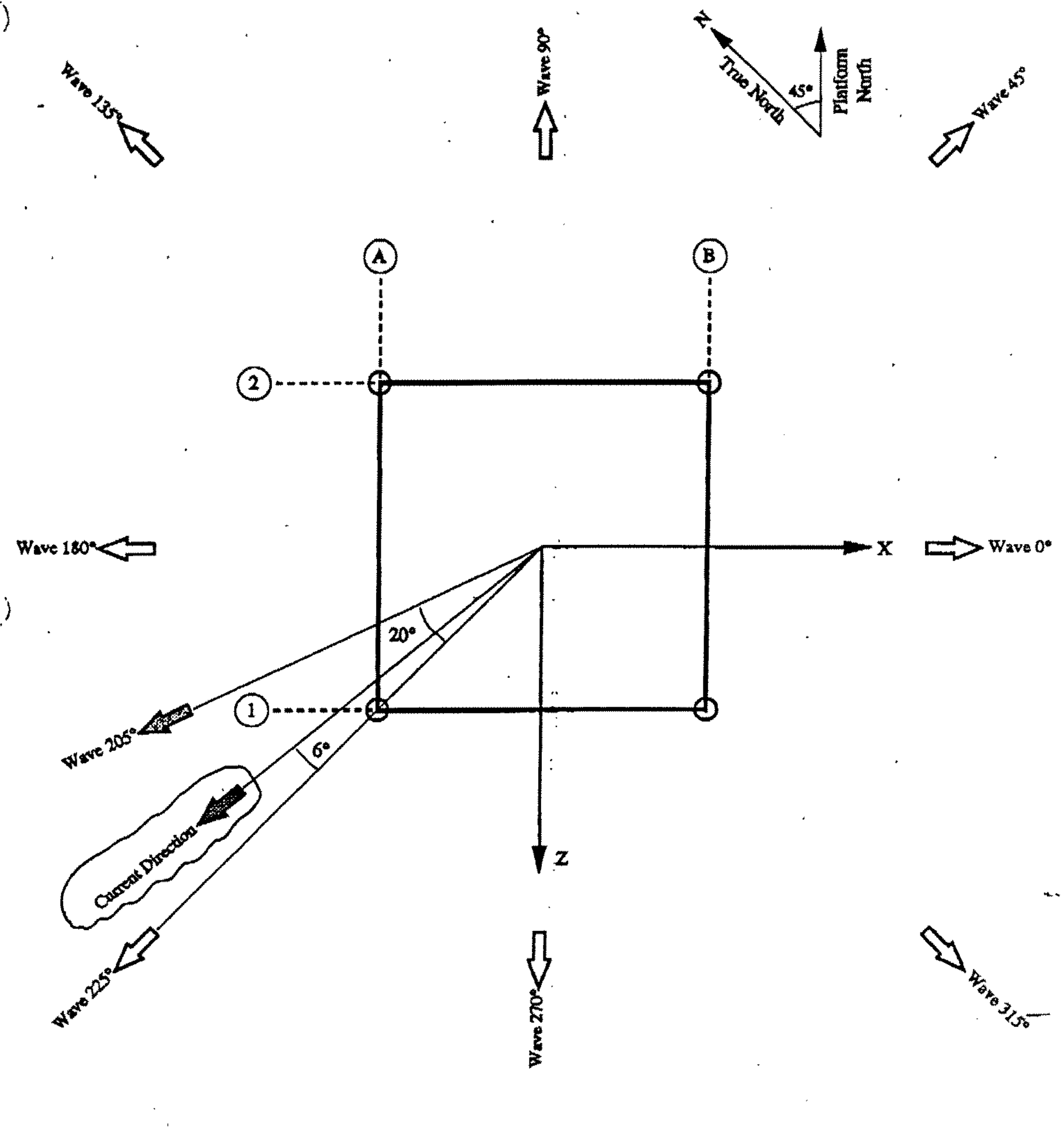


FIG. 1 **Platform Orientation & Wave Direction**

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.