HURRICANE ANDREW –
EFFECTS ON OFFSHORE PLATFORMS

Joint Industry Project

by

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INTRODUCTION

Hurricane Andrew passed through some 3,500 offshore structures in the Gulf of Mexico on August 24 to 26, 1992. While most structures were not adversely affected by Andrew, several sustained significant damage and in some cases collapsed. Recognizing that this type of experience provides a unique opportunity to further understand the performance of offshore structures in large storms, PMB solicited and gained support for a Joint Industry Project to study the effects of Andrew on offshore platforms. This is the final report, which documents the approach and results of the project.

The project had two primary objectives:

1. **Database.** Provide documentation on the offshore structures affected (survived, damaged or failed) by Hurricane Andrew. For damaged platforms, the emphasis was platforms with damage to primary load carrying structural members (miscellaneous damage to handrails, stairways, etc. was neglected).

2. **Calibration.** Perform a calibration of current procedures for assessing existing platforms (caissons excluded). The calibration involved comparison of "analytically" predicted platform damage and failure to "observed" platform performance during Andrew. The end result was a factor known either as a bias, calibration or correction factor which represents modeling uncertainties with respect to overall safety factor (resistance divided by loading effects) for platforms during hurricane loadings.

A secondary objective was to provide information to API Task Group 92-5 on "Assessment of Existing Platforms to Demonstrate Fitness for Purpose," which is currently drafting assessment procedures (some of which are similar to those used on this project) to be incorporated in a future edition of API RP 2A.

DATABASE

The database portion of the project involved gathering as much data as possible on what happened to platforms in Andrew. Data sources included participants, including direct input from the Minerals Management Service (MMS), cooperative non-participants and public documents. Meteorologic and oceanographic information for Andrew at each platform site was based upon a hindcast performed by Oceanweather Inc.

As the project progressed, it became apparent that the database would not be as comprehensive as originally anticipated. Other then direct input from participants, there was little "reliable" information about platform damage. In fact a large part of the effort was distinguishing fact from fiction, with just about every data source providing a different set of "damaged" platforms. Finally, it was decided to rely on input from the MMS as well as, where possible, direct confirmation regarding any platform damage with the platform owner. The final "best estimate" table of platforms and caissons damaged (significant structural or complete failure) in Andrew is shown in Table ES-1. Figure ES-1 shows the location of the platforms and caissons relative to the path of Andrew. Table ES-2 shows the "numbers" of damaged platforms and caissons. Table ES-3 shows the general trends of the data for platforms and caissons.

CALIBRATION

The calibration portion of the project involved first developing a "rigorous" analytical approach that could be used for the project. The approach determines what would happen to a specific platform in Andrew based upon analytical predictions, compares these results to what actually happened (observed) in Andrew, and then uses this information to establish a "bias factor, B" to improve the analytical procedures so that they are more consistent with actual observations. The bias factor could also be termed a calibration or correction factor and represents the modeling uncertainties with respect to the overall safety factor (resistance divided by loading effects) for platforms during extreme storm loadings. The favorable outcome for the bias factor, (i.e., "B" > 1) would indicate that the current platform checking process is conservative in the sense that more failures are predicted during storms than will actually occur. Drs. Allin Cornell (Stanford University) and Fred Moses (University of Pittsburgh) assisted in developing the analytical approach.

The steps in the calibration process were based upon "Bayesian" updating and were generally quite complex involving probabilistic computations. A summary of the approach is as follows:

1. Select representative platforms from the database for use in calibration. Table ES-4 lists the 13 platforms selected.

2. Develop a nonlinear computer model and perform static pushover analysis to determine the platform's capacity in a variety of loading directions (e.g. broadside, end-on and diagonal). The load and resistance modeling used in the process was based upon a "recipe" agreed with participants (primarily in accordance with the API RP 2A 20th edition).

3. Using a special probabilistic based computer code developed by the project, determine the probability of failure of the platform using load information from the Oceanweather hindcast and resistance information from the static pushover analysis. The intent was to develop "likelihood" curves for each platform that

provide the probability of failure of the platform during Andrew for different values of safety margin.

4. The likelihood curve is then used to establish the bias factor B as shown in Figure ES-2. The likelihood curve is multiplied by the initial or "prior" distribution for B (taken as a normal distribution with mean of 1.0 and uncertainty of 30 percent) to determine the final or "posterior" distribution of B. For an unexpected survival the distribution shifts to the right (assessment process is conservative), and for an unexpected failure the distribution shifts to the left (assessment process is unconservative).

5. The process is repeated for all platforms with the final "posterior" results as shown in Figure ES-3. Also shown are the "posterior" distributions considering individually survivals, failures and damage. The final posterior has a mean value of B=1.19, with an uncertainty of 10 percent. This implies that on average, for the platforms evaluated by this project, there is a 19 percent conservatism in the assessment "recipe" and pushover analysis technique used by the project.

Applying the bias factor to a single specific structure for purposes of "requalification," however, may lead to erroneous conclusions. The bias factor was averaged from a fleet of structures that were exposed to hurricane Andrew. These structures had a variety of potential platform failure modes with varying degrees of criticality. These included some subjectivity in interpreting the platform safety margins at specific locations. The exposed platforms may not be representative of any specific structure about which detailed predicted modal capacities and safety margins are available. Further studies (as recommended below) should be able to refine the bias factors for specific platform conditions and failure mode types.

CONCLUSIONS AND RECOMMENDATIONS

Database

The database indicates that a majority of the severely damaged or failed platforms were of 1960's or earlier vintage with most having incurred wave-in-the-deck loads during Andrew. The database was not as extensive as originally anticipated due to lack of data from non-participants. No steel jacket platforms designed to API RP 2A 9th edition (1977) or later were damaged or failed by Andrew metocean conditions (several post 1976 platforms were damaged and failed, but further investigation indicated these observations were due to pre-existing damage or impact from an out-of-control vessel). This is an important observation since API is currently considering design by the 9th edition as one of the criteria for platform reassessment. The variety of survived, damaged and failed platforms available from participants provided a good sampling of platform consequences for the calibration.

The primary recommended further work for the database includes gathering additional information on failures/survivals to support additional calibrations (see below).

Calibration

A favorable bias factor (B), with mean of 1.19 and COV of 0.10, established in this project reflects conservatism in load and resistance recipe, and pushover analysis technique followed in this project. This demonstrates that the existing process for platform assessment typically used by many operators (and used in this project) provides a realistic estimate of platform capacity.

As previously noted, it may be premature to use the bias factor for direct decisions on requalification; however, the direct application of the bias factor is justified in performing economic risk/cost/benefit remediation studies for fleets of older platforms. The favorable bias factor will therefore increase the average platform reliability to resist hurricanes beyond that computed by conventional analysis. Any global remediation decisions may be based on the updated reliabilities, which reflect the observed Andrew and other hurricane experiences.

The updated bias factor and the associated improved reliability estimates are potentially applicable also to the development of the new API requalification criteria. The application is legitimate for those cases when the criteria are explicitly based on a target risk level such as, for example, an annual failure rate of one per thousand. When the acceptance criteria are established by direct calibration with experience, however, the bias factor is implicitly reflected in such experience and cannot provide any further direct adjustments.

It is not known if this conservatism is due to estimates of load or resistance. As noted above, care should be taken in using the bias factor until further investigation is completed.

Primary recommended further work includes:

- Develop "multiple" bias factors – for example, a bias factor for platforms governed by brace failure and a bias factor for platforms governed by foundation failure. This project developed a "global" bias factor irrespective of failure modes.

- Develop and implement a process that uses a "weighting" procedure that accounts for the performance of other Gulf of Mexico platforms during Andrew that were not directly evaluated by the project. The intent is to increase in a simplified manner the number of platforms used to establish the bias factor.

- Investigate more directly component damage (braces, legs, joints, etc.) predicted analytically versus component damage actually observed. This project focused primarily on the platform capacity for use in calibration.

- Investigate more thoroughly the results of the nonlinear analysis and, where necessary, re-perform some of the structural analyses used in the calibration based on an updated recipe (i.e. different joint capacity equation).

- Investigate platforms in different categories (i.e., configuration, water depth), which were affected by past hurricanes, e.g., Hilda, Betsy, Camille, etc. Such an attempt combined with the weighting procedure would provide more representative multiple bias factors.

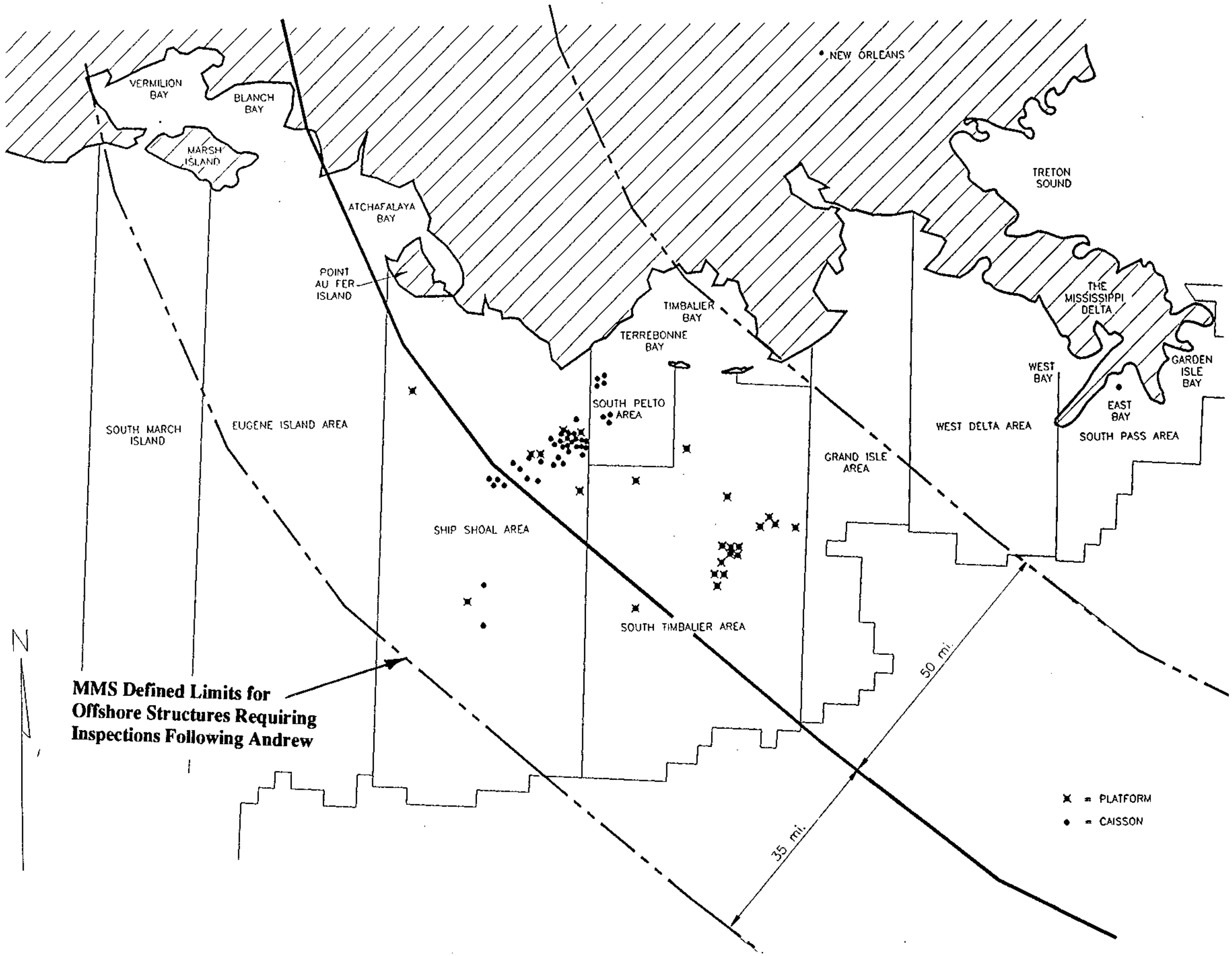


Figure ES-1 Path of Hurricane Andrew with Best Estimated Damaged Caissons and Platforms

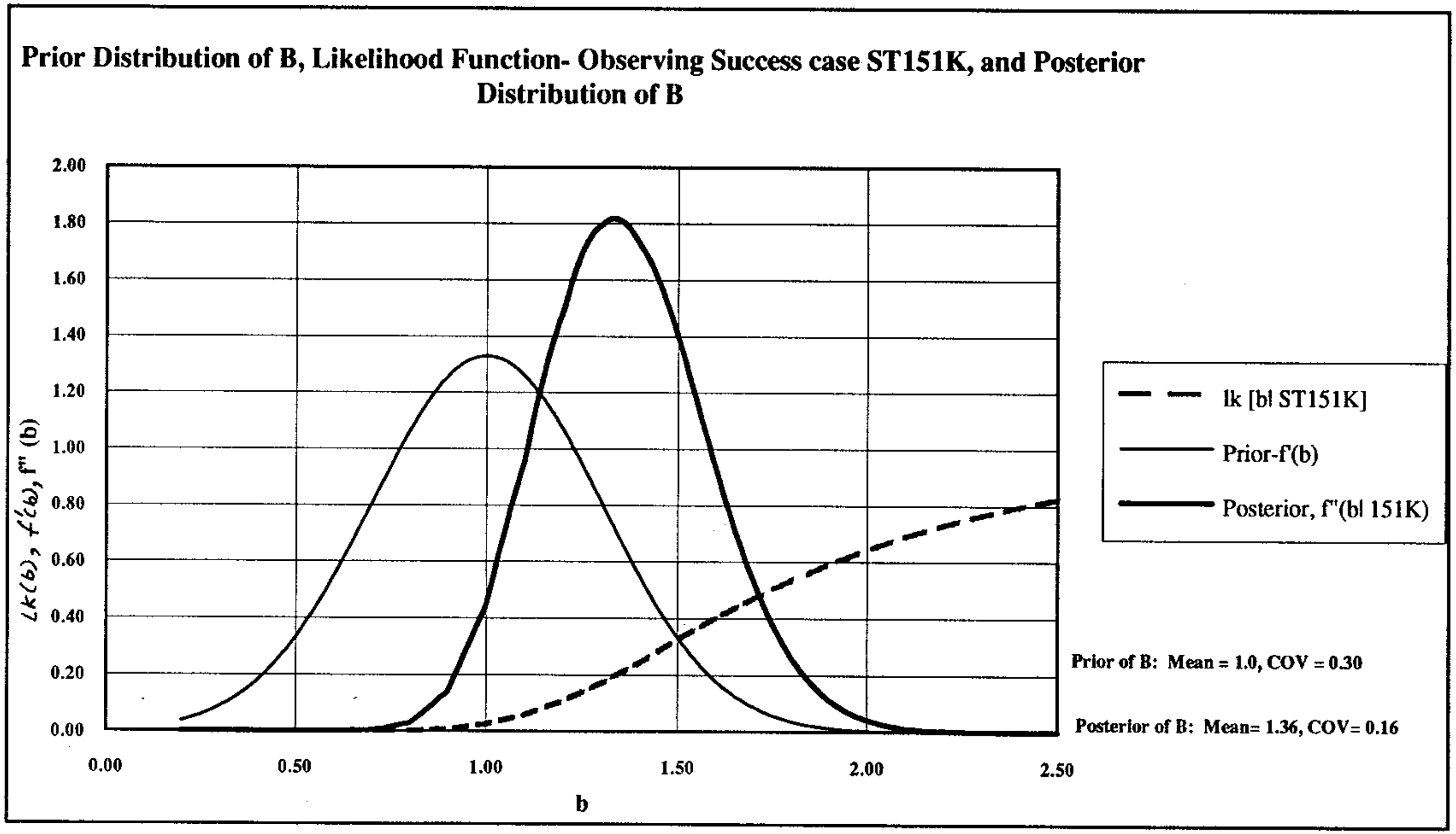


Figure ES-2 Likelihood Function and Posterior Distribution - Example Platform ST151K

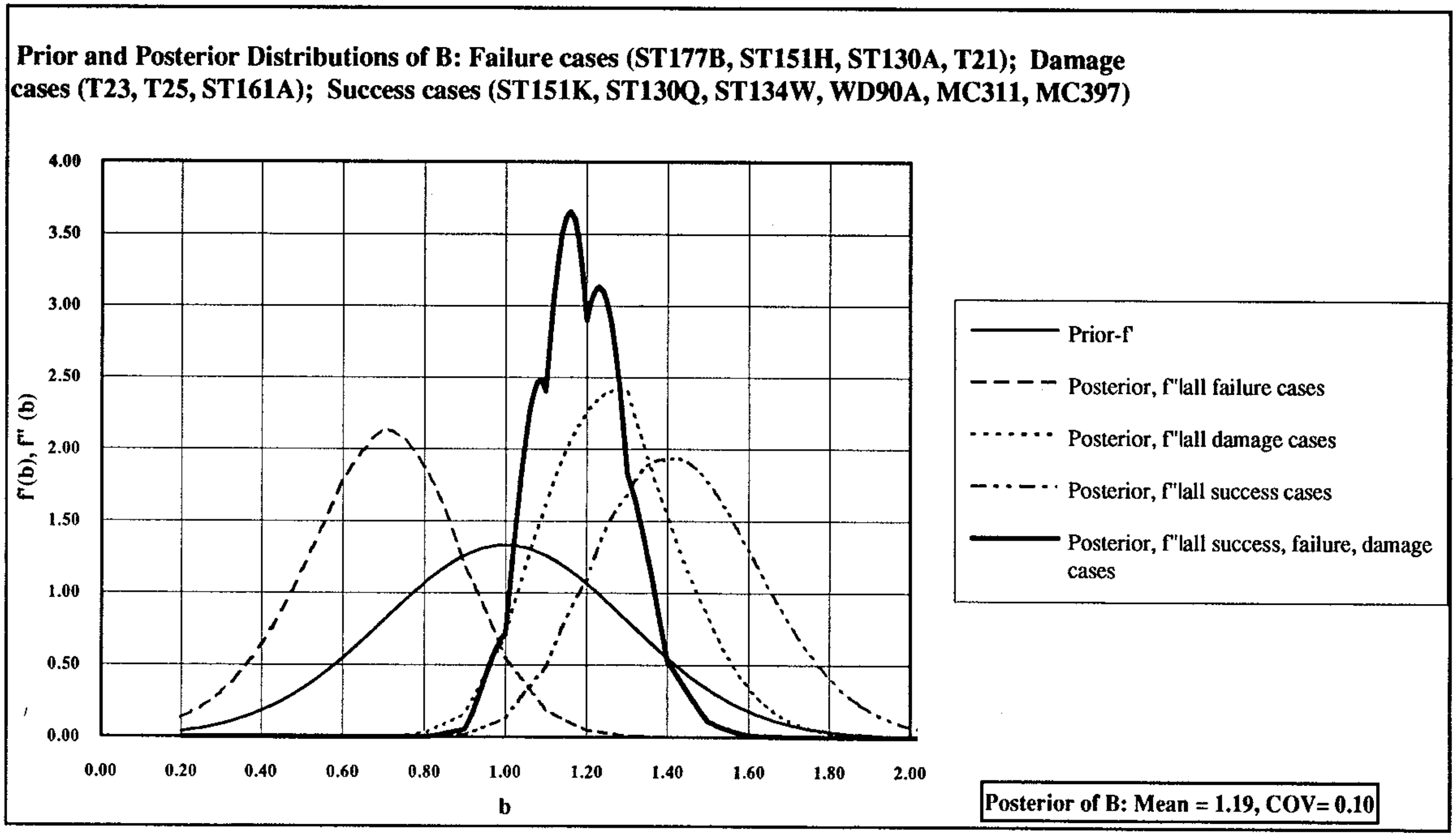


Figure ES-3 Posterior Distribution of Bias Factor (B) - Different Categories and All Cases Combined

Table ES-1 (a): Best Estimate of Steel Jacket Platforms and Caissons Damaged in Hurricane Andrew in OCS Waters

OPERATOR NAME	BLOCK	PLATFORM DESCRIPTION					MAX. SIG. WAVE HT. HURRICANE ANDREW (M)	DISTANCE OF PLATFORM FROM HURRICANE EYE (MILES)	PLATFORM DAMAGE DESCRIPTION	DAMAGE CONFIRMED BY OPERATOR
		NAME OR WELL NO.	TYPE	WATER DEPTH (FEET)	YEAR INSTALLED	DISTANCE FROM SHORE (MILES)				
AMOCO	ST 0161	A	Platform	112	64	32	11	5	Horizontal Punched into Legs; Cracks in K-joints Yielding in 4-K joints	Yes
CHEVRON USA INC	ST 0130	A	Platform	140	58	28	11	21	Toppled	Yes
	ST 0134	F	Platform	137	62	29	11	18	Abandoned, Buckled, and Cracked Vertical Diagonals at Severel Elevations; to be Removed	Yes
	ST 0134	W	Platform	137	81	29	11	18	Buckled Member at Waterline; No longer Aligned	Yes
	ST 0135	M	Platform	116	66	29	11	15	Horizontal X-brace Cracked; to be Removed	Yes
	ST 0151	H	Platform	137	64	32	11	14	Toppled	Yes
	ST 0151	J	Platform	140	62	32	11	14	Many Damaged Members Near Mudline; Being Repaired	Yes
	ST 0151	L	Platform	140	66	32	11	14	Vertical X-brace Cracked; to be Removed	Yes
	ST 0151	PROD-1	Platform	137	62	32	11	14	2 Vertical X-braces Damaged; 1 Replaced, 1 Repaired	Yes
	ST 0152	E	Platform	137	60	32	11	14	Toppled	Yes
	ST 0177	B	Platform	142	65	35	11	14	Sverely Damaged; being Removed	Yes
	ST 0177	B AUX	Platform	142	65	35	11	14	Sverely Damaged; being Removed	Yes
	ST 0188	CA	Platform	143	81	38	11	5	Deck Severely Damaged; Deck Shimmed, not Grouted; Deck had Large Cantilever; being Replaced; Jacket Undamaged	Yes
KERR-MCGEE CORPORATION	SS 0230	A	Platform	120	62	42	8	18		No
MOBIL OIL EXPLORATION & PRODUCTION	SS 0072	A	Platform	29	48	6	6	8	Total Loss, Collapsed; all Wells Abandoned Deck Broke Off; Jacket Broke Off 2' Above Mudline	Yes
	PL 0009	#4	Caisson	40	80	6	8	80		Yes
	PL 0009	#9	Caisson	40	85	6	8	80	Toppled	Yes
	PL 0010	#17	Caisson	40	84	5	8	77	Leaning 12 Deg.	Yes
	PL 0010	#18	Caisson	35	87	7	8	77	Toppled	Yes
MURPHY E&P	PL 0020	#13	Platform		67					Yes
	SS 0093	#6	Platform		69					Yes
	SS 0114	HEADER	Platform	54	65	13	6	8	Toppled	Yes
	SS 0114	#41	Platform	41	63	13	6	8	Leaning 30 Deg.	Yes
	SS 0118	E	Platform	54	54	16	6	4	Toppled; Heavy Rainbow Slick 2 MI by 800 YDS	Yes
	SS 0119	#5	Platform	43	73	16	6	3	Leaning 10 Deg.	Yes
	ST 0086	C	Platform	92	76	21	8	24	Pipeline Junction Platform T-21 Toppled; Hit by MODU	Yes
	PL 0019	#14	Caisson	30	81	12	8	80	Leaning 25 Deg.	Yes
	PL 0019	#33	Caisson		81					Yes
	PL 0019	#48	Caisson		86					Yes
	PL 0020	#28/#29	Caisson		81					Yes
	PL 0020	#33	Caisson		85					Yes
	PL 0020	#40/#42	Caisson		86					Yes
	SS 0090	#7	Caisson		91					Yes
	SS 0093	#12	Caisson		77					Yes
	SS 0093	53	Caisson	32	87	10	7	10	Toppled	Yes
	SS0094	#23	Caisson	48	87	20	7	9	Leaning 5 Deg.	Yes
	SS 0112	#4	Caisson	48	86	20	6	4		Yes
	SS 0112	#5	Caisson	47	86	20	6	4	Leaning 6 Deg.	Yes
	SS 0112	#6	Caisson	48	87	19	6	4	Leaning 5 Deg.	Yes

Table ES-1 (b): Best Estimate of Steel Jacket Platforms and Caissons Damaged in Hurricane Andrew in OCS Waters

OPERATOR NAME	BLOCK	PLATFORM DESCRIPTION					MAX. SIG. WAVE HT. HURRICANE ANDREW (M)	DISTANCE OF PLATFORM FROM HURRICANE EYE (MILES)	PLATFORM DAMAGE DESCRIPTION	DAMAGE CONFIRMED BY OPERATOR
		NAME OR WELL NO.	TYPE	WATER DEPTH (FEET)	YEAR INSTALLED	DISTANCE FROM SHORE (MILES)				
MURPHY E&P	SS 0113	#37	Caisson	50	81	13	6	7		Yes
	SS 0113	#41	Caisson	41	85	13	6	7		Yes
	SS 0113	#45	Caisson	45	90	15	6	7	Toppled	Yes
	SS 0113	#47	Caisson	47	89	18	6	7	Leaning 48 Deg.	Yes
	SS 0113	#48	Caisson	47	89	18	6	7	Toppled	Yes
	SS 0113	#51	Caisson	47	92	13	6	7	Leaning 15 Deg.	Yes
	SS 0113	#52	Caisson	47	90	13	6	7	Leaning 5 Deg.	Yes
	SS 0113	N	Caisson	48	88	13	6	7		Yes
	SS 0114	#1	Caisson	54	63	13	6	8	Toppled	Yes
	SS 0114	#46	Caisson	50	83	18	6	8	Leaning 35 Deg.	Yes
	SS 0114	#50	Caisson	46	84	19	6	8	Toppled	Yes
	SS 0114	#51	Caisson	50	82	13	6	8	Leaning 30 Deg.	Yes
	SS 0114	#55	Caisson	40	81	13	6	8	Leaning 45 Deg.	Yes
	SS 0114	#57	Caisson	50	86	18	6	8	Toppled	Yes
	SS 0114	#60	Caisson	51	85	13	6	8	Toppled	Yes
	SS 0114	#61	Caisson	53	86	18	6	8	Leaning 10 Deg.	Yes
	SS 0117	#2	Caisson	50	81	18	6	6	Leaning 30 Deg.	Yes
	SS 0117	#3	Caisson	53	82	18	6	6	Leaning 20 Deg.	Yes
	SS 0117	#7	Caisson	55	90	15	6	6	Leaning 45 Deg.	Yes
	SS 0118	#9	Caisson	53	92	21	6	4	Toppled	Yes
	SS 0119	Q	Caisson		82	16	6	3		Yes
	SS 0120	#2	Caisson	35	83	18	6	1	Leaning 5 Deg.	Yes
	SS 0120	#4	Caisson	40	83	17	6	1	Leaning 15 Deg.	Yes
	SS 0134	#20	Caisson	60	85	20	7	1	Toppled	Yes
	SS 0134	#7	Caisson	60	83	20	7	1	Leaning 45 Deg., Structure Repaired	Yes
	SS 0135	#10	Caisson	113	83	20	7	1	Leaning 10 Deg., Structure Repaired	Yes
	SS 0135	#5	Caisson	45	82	22	7	1	Leaning 5 Deg.	Yes
	SS 0136	#1	Caisson	65	83	32	7	2	Leaning 30 Deg.	Yes
	SS 0136	#2	Caisson	70	87	32	7	2	Leaning 20 Deg.	Yes
	ST 0086	#20	Caisson	110	83	22	8	24		Yes
SAMEDAN OIL CORPORATION	ST 0172	A	Platform	107	64	37	10	1	Platform Leaning 20 Deg., 360 Deg. Tear NE Jacket Leg, 2 Deck Legs Sheared, Rest Bent; SE Pile Pullout 15'; Adjacent East Pile Pullout 5'	Yes
STONE PETROLEUM CORPORATION	SP 0023	CA	Platform	61	62	15	6	70		No
TRUNKLINE	ST 0072	T-21	Platform	61	69	18	8	12	Toppled	Yes
TRUNKLINE	ST 0052	T-23	Platform	63	69	18	8	12	Damaged	Yes
TRUNKLINE	SS 0139	T-25	Platform	62	69	18	8	12	Heavily Damaged	Yes
UNOCAL	SS 0215	C	Platform	110	62	37	8	14	Toppled, Pre-existing Damage Scheduled for Removal Prior to Andrew	Yes

Table ES-2: Summary of Damaged Steel Jacket Platforms

Gulf of Mexico Area	Year Installed	Water Depth (ft.)	Distance of Platform from Hurricane Eye (Miles)	Number of Platforms	Condition of Platforms
South Timbalier (ST)	1948 - 1969	up to 100'	12	2	5 Toppled 6 Extensively damaged- being removed 5 with minor to heavy damage 1 with deck severly damaged- being replaced
		101' - 150'	1 to 5	2	
			14 - 21	10	
	1970 - 1977	up to 100'	24	1	
	1978 - 1981	101' - 150'	5, 18	2	
Ship Shoal (SS)	1948 - 1969	up to 50'	8	2	4 Toppled 3 with moderate to heavy damage 2 with unknown damage
		51' - 100'	4 to 12	3	
		101' - 150'	14, 18	2	
	1970 - 1977	43'	3, 24	2	
South Pass (SP)	1962	61'	70	1	Unknown damage
South Pelto (PL)	1967	unknown	unknown	1	Unknown damage
			Total	28	

Table ES-3: General Trends - Damaged Steel Jacket Platforms and Caissons

Item	Units	Damaged Platforms and Caissons	Damaged Platforms Only
Average Water Depth	ft.	69.1	101.2
Average Age	years	17	30
Average Distance to Center Path of Storm	miles	14.6	13.6
Average Distance from Shore	miles	20.2	25.4
Average Hs	ft.	24.6	29.2
Oldest Damaged Structures (year installed)	-	SS072A (1948) * SS118E (1954) * ST130A (1958) *	SS072A (1948) SS118E (1954) ST130A (1958)
Newest Damaged Strutures (year installed)	-	SS0113#51 (1992) ** SS0118#9 (1992) ** SS090#7 (1991) **	ST134W (1981) ST188-CA (1981) ST086C (1976)

Notes : * Damaged Steel Jacket Platform
** Damaged Caisson

Table ES-4: Platforms Selected for Calibration - Andrew JIP

Platform Operator	Platform Name	Platform Location- Block	Water Depth ft.	Year of Installation	Number of legs	Performance in Andrew	Pushover Analysis Performed by
Survival Platform Cases:							
Chevron	ST151K	South Timbalier-151	137	1963	8 leg	Survived	PMB
Chevron	ST130Q	South Timbalier-130	170	1964	4 leg	Survived	PMB
Chevron	ST134W	South Timbalier-134	137	1981	4 leg	Damaged *	PMB
Amoco	WD90A	West Delta-90	184	1964	8 leg	Survived	PMB
Exxon	MC311	Mississippi Canyon-311	343	1978	8 main legs, 8-skirt legs	Survived	Exxon
Exxon	MC397	Mississippi Canyon-397	468	1991	4 main legs, 4-skirt legs	Survived	Exxon
Damage Platform Cases:							
Trunkline	T23ST52	South Timbalier-52	63	1969	4 leg	Minor Damage	PMB
Trunkline	T25SS139	Ship Shoal-139	62	1969	4 leg	Heavily Damaged	PMB
Amoco	ST161A	South Timbalier-161	118	1964	8 leg	Minor Damage	Amoco
Failure Platform Cases:							
Chevron	ST177B	South Timbalier-177	142	1965	8 leg	Heavily Damaged/ Salvaged **	PMB
Chevron	ST151H	South Timbalier-151	137	1964	8 leg	Collapsed	PMB
Chevron	ST130A	South Timbalier-130	140	1958	8 leg	Collapsed	Chevron
Trunkline	T21ST72	South Timbalier-72	61	1969	4 leg	Collapsed	PMB

Notes:
* ST134W Platform considered as a survival case, as analysis did not reflect that hurricane alone could have caused the observed damage.
** ST177B platform was heavily damaged in Andrew. This platform has now been removed and salvaged.

Section 1
Introduction

1.1 BACKGROUND

Hurricane Andrew (hereafter called Andrew) was a very intense storm that passed through the Gulf of Mexico on August 24, 25 and 26, 1992. In particular, Andrew passed through a region of some 3,500+ offshore structures located offshore Louisiana. While most of these platforms were not adversely impacted by Andrew, several suffered problems ranging from minor damage such as bent handrails, to severe damage such as a buckled underwater brace, to catastrophic damage such as complete collapse of the structure.

In addition, assessment of existing platforms is receiving considerable attention in U.S. waters due to the age of many platforms. The American Petroleum Institute (API) is currently drafting procedures for assessment of existing platforms that will be included in a future edition of API RP 2A. Platform assessment is also receiving attention in the North Sea due to issues associated with Safety Case Evaluations.

An extreme event such as Andrew provides an opportunity to learn from the experience by reviewing the platforms that survived, were damaged, or failed during the hurricane and trying to understand what happened and why. It is a unique opportunity to study offshore structures tested under full scale real conditions.

Based upon this background, PMB solicited and gained support for a Joint Industry Project (JIP) with two goals. The first was to collect as much data as possible about Andrew's impact on offshore platforms. The primary emphasis would be on platforms that survived the most severe portions of the storm, suffered damage to key structural elements, or collapsed. The information would be combined into a "database" and be used as a basis for the calibration work (second goal).

The second and more important goal was to perform a "calibration" of current industry practice for assessment of existing platforms. This effort would include structural analysis of several platforms (selected from the database described above) in order to predict on an analytical basis what should have happened to the platforms during Andrew. These analytical results would be compared to actual "observed" events from Andrew. A rigorous mathematical calibration procedure (Bayesian updating) would then be used to determine a "bias" factor that could be applied to the analytical process so that analytical results more closely agree with observed results.

1.2 OBJECTIVES

There were two primary objectives for the Hurricane Andrew JIP:

1. **Database.** Provide documentation on the primary platforms affected (survived, damaged or failed) by Andrew. Only steel jacket platforms were considered (although some limited information was provided for caissons). Although this is intended as a general database about what happened to platforms during Andrew, the primary intent was to provide data for the calibration effort.

2. **Calibration.** Perform a calibration of procedures for assessing existing platforms. A rigorous method for calibration was used to ensure as accurate results as possible. The process includes comparisons of analytically predicted platform damage and failure to observed field performance during Andrew.

A secondary objective was to provide information to API Task Group (TG) 92-5 on "Assessment of Existing Platforms to Demonstrate Fitness For Purpose" which is currently drafting reassessment procedures to be incorporated into API RP 2A. This project provided information and results to the TG for testing of several proposed analytical procedures.

1.3 PROJECT PARTICIPANTS

The project was jointly funded by 12 regulators and operators. The participants are as follows including the primary technical representative for each.

AMOCO	– Mr. Gary Imm
BRITISH PETROLEUM	– Mr. John Kleinhans
CHEVRON	– Mr. Dirceu Botelho
EXXON	– Mr. Ward Turner
HEALTH AND SAFETY EXECUTIVE (MATSU)	– Dr. W. J. Supple
MINERALS MANAGEMENT SERVICE	– Mr. Charles Smith
MOBIL	– Mr. David Petruska
PHILLIPS	– Mr. Roger Thomas
SHELL	– Mr. Kris Digre
TEXACO	– Mr. Dave Wisch
TRUNKLINE	– Mr. Jim Meyer
UNOCAL	– Mr. Jared Black

1.4 PROJECT TEAM

PMB Engineering Inc. was the prime contractor, providing all project management, analysis (structural and calibration) and reporting. Key PMB staff were as follows including their principle work tasks:

Project Management	– Mr. Frank Puskar
Calibration	– Dr. Rajiv Aggarwal
Structural Analysis	– Ms. Margaret Longstreth

In addition, the following provided consulting services throughout the project:

Dr. Allin Cornell, Stanford University – input to calibration and structural analysis. Review and comment of results.

Dr. Fred Moses, University of Pittsburgh – input to calibration and structural analysis. Review and comment of results.

Mr. Griff Lee, Griff Lee Inc. – assistance with the sources for platform information

Meteorologic and oceanographic site specific hindcast information for Hurricane Andrew was provided by Oceanweather Inc. under an agreement with PMB. Participants were provided a copy of the Oceanweather Hurricane Andrew Hindcast as a deliverable for the project. Participants already owning the Oceanweather Hurricane Andrew Hindcast received an equivalent credit on the participation fee for this project.

1.5 ACKNOWLEDGMENTS

Several participants contributed significant effort to ensure that this project was successful. In particular, the following should be commended for their input:

- **CHEVRON.** Chevron provided a significant number of the platforms (for calibration) and the project would not have been possible without their contribution. Chevron also provided analytical results (nonlinear pushover analysis) from in-house analyses. Chevron personnel providing assistance included Dirceu Botelho, Bill Krieger, Chuck Petrauskas, David Kan, and Mani Vannan.

- **EXXON.** Exxon provided analytical results (nonlinear pushover analysis) for two complex platforms. Exxon also provided technical information regarding soil properties that should be used for pushover analyses. Exxon individuals providing assistance included Ward Turner, Hugh Banon, Don Murff and Jim Alexander.

- **AMOCO.** Amoco provided descriptive information for one platform for use by PMB in structural analysis and also provided analytical results (nonlinear pushover analysis) for another platform. Amoco personnel providing assistance included Gary Imm and James Light.

- **TRUNKLINE.** Trunkline provided descriptive information for three platforms for use by PMB in structural analysis. Trunkline personnel providing assistance included Jim Meyer and John Alholm.

- **MINERALS MANAGEMENT SERVICE.** The MMS provided the project with several updates to the overall Hurricane Andrew database. MMS personnel providing assistance included Charles Smith and Felix Dyhrkopp.

Mr. Kris Digre of SHELL provided assistance in communication with the API Task group. Dave Wisch of TEXACO, Mr. Malcolm Sharples of NOBLE DENTON, and Mr. Paul Fourchy of MURPHY OIL provided assistance with information for the Hurricane Andrew database.

Section 2
Database

2.1 APPROACH

Information for the database was gathered from a variety of sources with emphasis on damage to primary structural members of steel jackets. Platforms with damage to secondary structural elements such as handrails and walkways were not included. An effort was also made to identify platforms that survived some of the most extreme conditions of Andrew, with emphasis on "unexpected survivals" that would have the largest impact on the calibration.

Primary sources for information were as follows:

Participants owning affected platforms
Minerals Management Service
API Information Exchange (October 29, 1992)
Cooperative non-participants owning affected platforms
Noble Denton
Public Sources

PMB accumulated and then sorted through all of the information provided by these sources. The information was condensed and placed in a summary spreadsheet (Microsoft Excel) with data sorting capabilities. The hardcopy and electronic database were provided to all participants as part of the study.

Several case studies were developed for a number of platforms where additional information was available. This included both qualitative case studies (a brief description of what happened) contained in this section, and quantitative case studies where detailed structural analysis was performed as part of the calibration effort, described in Section 3.

Generally, the project was unable to locate as much information about damaged platforms as originally anticipated. Cooperation with project participants was excellent, but obtaining information from non-participants was difficult. All non-participant companies that appeared to have damaged structures were contacted, with some providing information, others indicating they may provide information (but ultimately not providing information) while others indicating that the information was unavailable.

From the onset of the project, the database was a secondary task compared with the calibration effort. Thus there was a limited amount of time and budget available to pursue further detailed information. However, the database did achieve its primary goal of obtaining a representative sample of affected offshore platforms which could be used as a basis for selection of platforms for calibration.

In addition, as requested by several participants, the Andrew database was also updated (using information from the AIM III project [PMB, 1988]) to include platforms that failed during previous hurricanes in the Gulf of Mexico.

2.2 HURRICANE ANDREW HINDCAST

The hindcast metocean (meteorological and oceanographic) conditions used in this project were taken from the Hurricane Andrew Hindcast performed by Oceanweather Inc. Metocean data was extracted from the hindcast for each of the quantitative case studies. The complete hindcast is available in a separate document [Oceanweather, 1992] provided to participants via this project or Oceanweather.

Figure 2-1 shows the best estimate path of the eye of Andrew (based upon the Oceanweather report) superimposed on the Gulf of Mexico offshore lease blocks. Figure 2-2 shows the path of Andrew along with the damaged platforms and caissons described in Table 2-1. Also shown is the MMS defined region of offshore structures requiring inspection. Figure 2-3 shows the path of Andrew with the platforms used in the calibration process described in Section 4.

As an additional part of this project, Oceanweather summarized the differences between tropical cyclones such as Andrew which affect the Gulf of Mexico and extratropical cyclones which affect the North Sea. The intent was to provide some discussion on how large storms differ between these two regions with a high density of offshore platforms. This discussion can be found in Appendix D.

2.3 DATABASE SUMMARY

Table 2-1 shows the "best estimate" list of platforms and caissons located in the MMS regulated outer continental shelf (OCS) region that were significantly damaged in Andrew. This "best estimate" is based upon information provided by the MMS plus direct confirmation by PMB with several owners of the platforms. Caissons were included to indicate the overall number of offshore structures affected. Only caissons which lean ±5 degrees or greater (defined as a "damaged" caisson per the MMS) were included. The individual structures confirmed by PMB are indicated in the table.

Tables 2-2 and 2-3 provide a summary of the numbers of platforms and caissons damaged and/or collapsed in Andrew, respectively, based upon Table 2-1. The total number of 75 affected offshore structures is lower than that provided in the project kickoff meeting (150+ total affected) or interim meeting (124 total affected). The number is significantly lower than indicated in earlier public reports immediately following the storm, such as the New York Times article of October 21, 1992 which indicated 249 damaged platforms. Note that many of these public reports probably reflect all types of damage (e.g. bent handrails and walkways) not included here.

Table 2-2 provides a summary of damaged and collapsed steel jacket platforms during Andrew. From this table, it is noted that a total of 28 steel jacket platforms installed in a water depth of less than 150 ft were damaged or collapsed. Most of these platforms were located in South Timbalier (ST) and Ship Shoal (SS) blocks at a distance up to 24 miles from the eye of the hurricane. Twenty-three platforms were installed before 1969, three were installed between 1970 and 1977, and only two were installed after 1977. Of these, at least 15 structures are reported to have toppled or were extensively damaged and are being removed. One damaged platform, installed in 1962, was located in the South Pass (SP) area in 61 ft of water, 70 miles from the hurricane eye.

Table 2-3 provides a summary of 47 caisson structures damaged and collapsed during Andrew. Most of this damage occurred in water depths of 50 ft or less, and only two damaged caissons were in water depths greater than 100 ft. Note that, with one exception, all the caissons were installed after 1977 and some of the damaged caissons were installed in 1992. Most of the damage was in the Ship Shoal blocks, with caissons located within 1 to 10 miles of the hurricane eye. In the South Timbalier blocks, only one caisson was damaged; it was located in 100 ft of water 24 miles from the hurricane eye. In the South Pelto (PL) blocks, the damaged caissons were located roughly 80 miles from the hurricane eye.

Table 2-4 provides a summary of the general trends of the platforms and caissons. In terms of platforms (which are the focus of this study), it is seen that only three platforms (ST 86C, ST 188, ST 134W) installed after 1976 were severely damaged or failed. However, these damage/failures are believed to be caused by pre-existing conditions or causes other than large waves. ST 86C was impacted by a MODU that broke free during the hurricane. ST 188 suffered loss of the deck which was apparently not fully grouted to the jacket during construction. ST 134 W suffered a buckled brace near the waterline which may have been caused by pre-existing damage (e.g., jackup impact).

This is an important observation since 1977 is approximately the time frame when the wave loading criteria specified in API RP 2A was significantly upgraded (9th edition). The database indicates that all platforms designed to the 9th edition survived Andrew (a 100 to 150 yr return period storm) with little or no damage. Design by the 9th edition or later of API RP 2A is one of the platform reassessment criteria being considered by API TG 92-5.

Table A-9 (Appendix A) shows a list of platforms surviving (without significant damage) some of the more intense conditions of Andrew and that were located in the MMS defined region of platforms requiring post-Andrew inspection (50 miles to the northeast and 35 miles to the southwest of the hurricane path). The list indicates the large number of platforms surviving Andrew with little or no damage. The list served as a basis during the project to select "surviving" platforms, particularly those that were "unexpected" survivals, for use in the calibration exercise.

As requested by several participants, Table 2-5 shows a list of Gulf of Mexico platforms that failed in hurricanes prior to Andrew. This information is taken from the previous AIM III project [PMB 1988].

2.4 QUALITATIVE CASE STUDIES

The qualitative case studies consisted of more in-depth reporting of selected platforms from the database lists. The focus was "interesting" platforms that portrayed an unusual situation or conveyed an important point. There were a total of eight qualitative case studies, including platforms that survived, were damaged or collapsed during Andrew.

Table 2-6 summarizes the qualitative case studies. Included is the platform name, background information, damage summary (if any) and general reason the platform was selected. Tables 2-7 and 2-8 provide additional information for two of the platforms. Appendix A (Table A-1 to Table A-8) contains similar information for all eight platforms.

It was initially anticipated that there would be more qualitative case studies and that they would be more in-depth; however, it became apparent that detailed information was often lacking, and unless confirmed by the platform owner, the project did not want to include the information in this report. In addition, several of the originally planned qualitative case studies were eventually converted to capacity assessments, where the project included a greater number of cases than originally anticipated (Section 3).

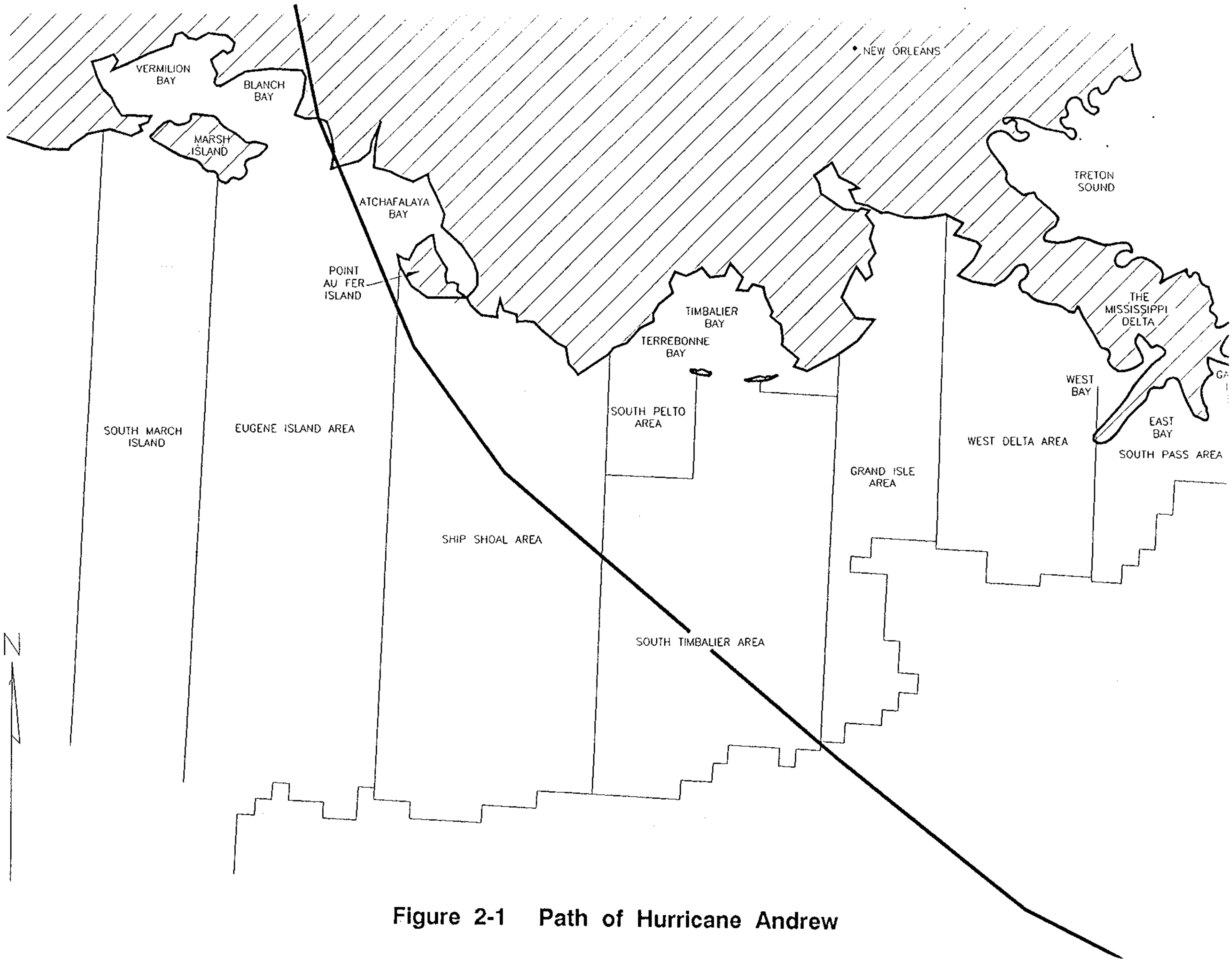


Figure 2-1 Path of Hurricane Andrew

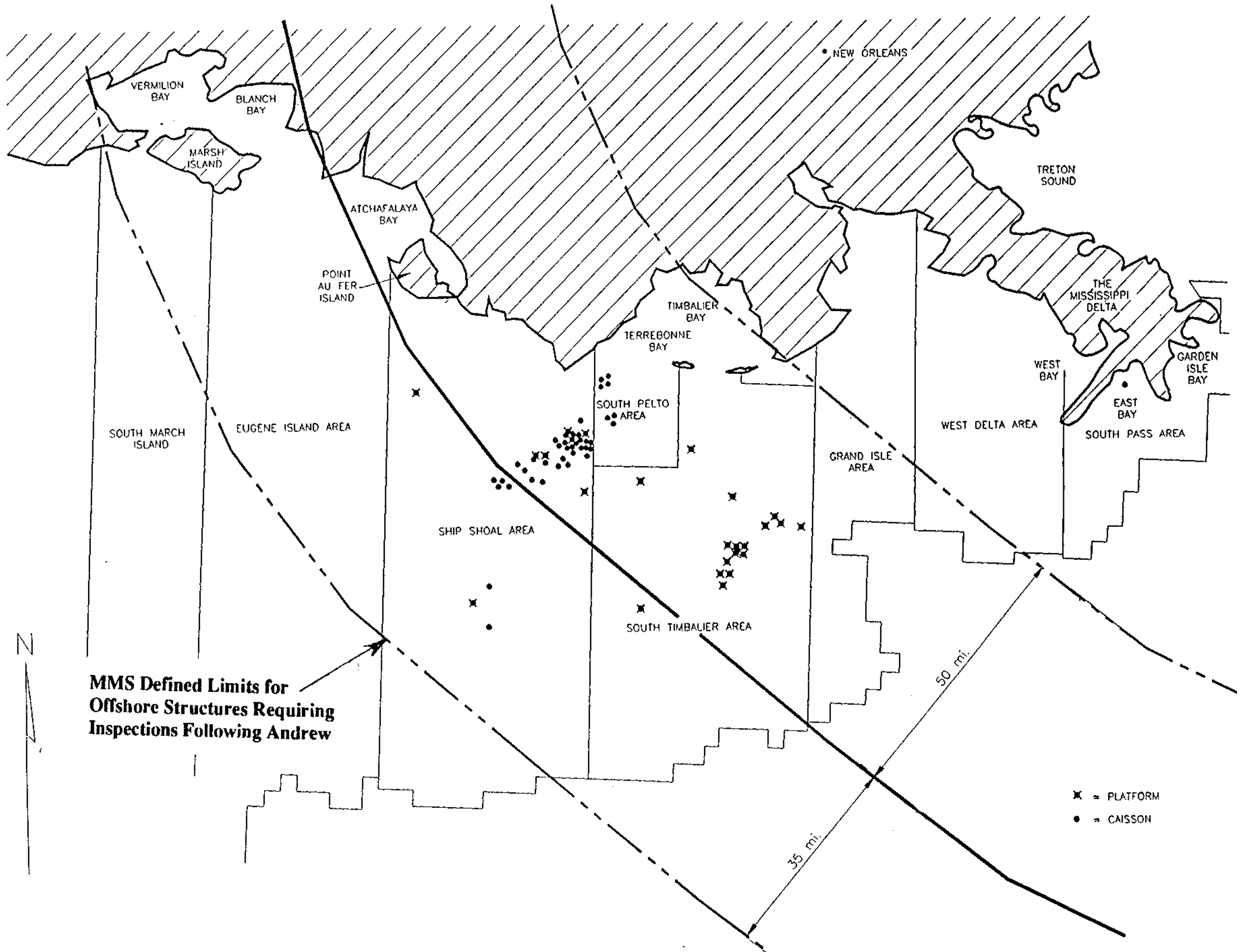


Figure 2-2 Path of Hurricane Andrew with Best Estimated Damaged Caissons and Platforms

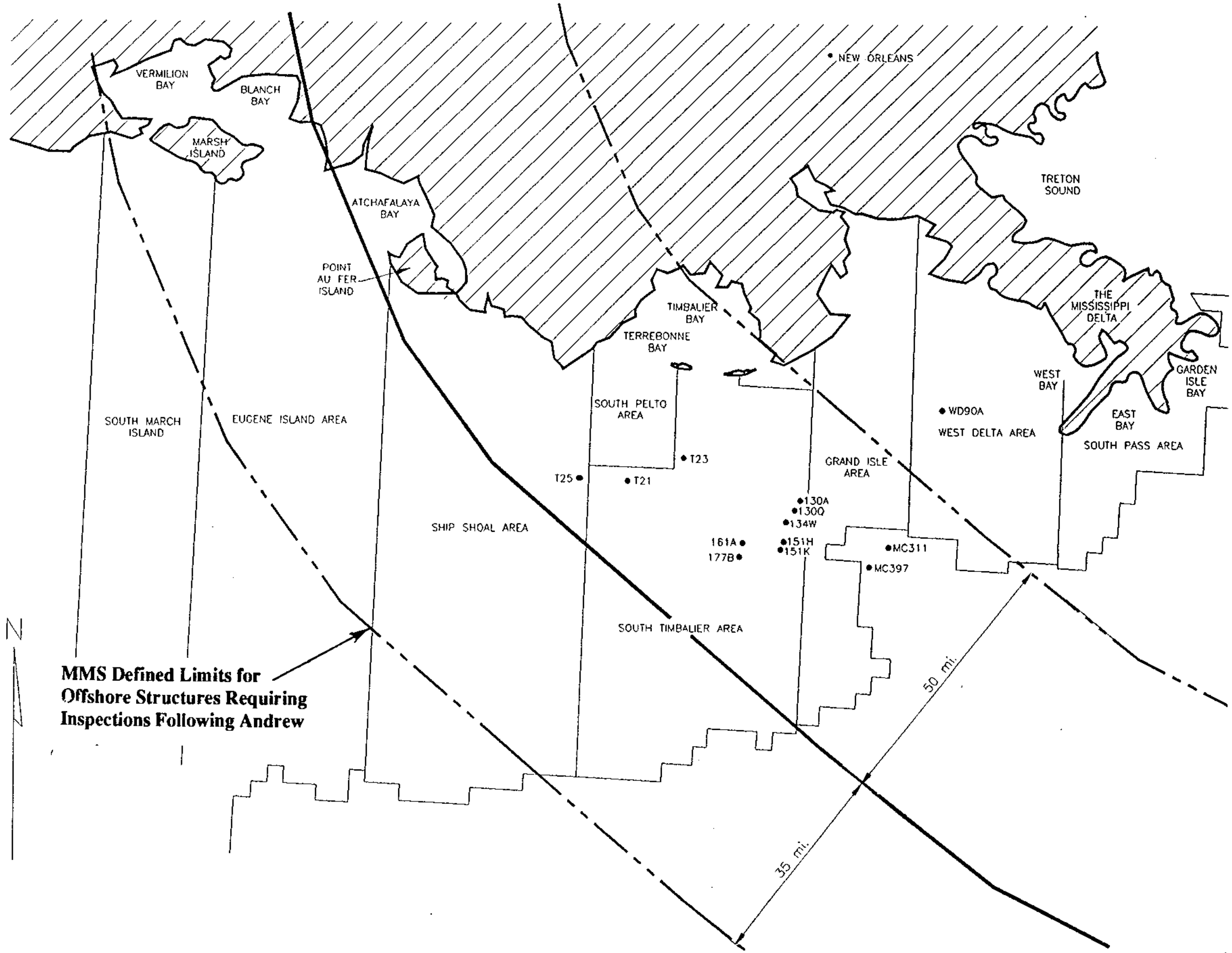


Figure 2-3 Path of Hurricane Andrew with Platforms Used in Calibration

Table 2-1 (a): Best Estimate of Steel Jacket Platforms and Caissons Damaged in Hurricane Andrew in OCS Waters

OPERATOR NAME	BLOCK	PLATFORM DESCRIPTION: NAME OR WELL NO.	TYPE	WATER DEPTH (FEET)	YEAR INSTALLED	DISTANCE FROM SHORE (MILES)	MAX. SIG. WAVE HT. HURRICANE ANDREW (M)	DISTANCE OF PLATFORM FROM HURRICANE EYE (MILES)	PLATFORM DAMAGE DESCRIPTION	DAMAGE CONFIRMED BY OPERATOR
AMOCO	ST 0161	A	Platform	112	64	32	11	5	Horizontal Punched into Legs; Cracks in K-joints Yielding in 4-K joints	Yes
CHEVRON USA INC	ST 0130	A	Platform	140	58	28	11	21	Toppled	Yes
	ST 0134	F	Platform	137	62	29	11	18	Abandoned, Buckled, and Cracked Vertical Diagonals at Severel Elevations; to be Removed	Yes
	ST 0134	W	Platform	137	81	29	11	18	Buckled Member at Waterline; No longer Aligned	Yes
	ST 0135	M	Platform	116	66	29	11	15	Horizontal X-brace Cracked; to be Removed	Yes
	ST 0151	H	Platform	137	64	32	11	14	Toppled	Yes
	ST 0151	J	Platform	140	62	32	11	14	Many Damaged Members Near Mudline; Being Repaired	Yes
	ST 0151	L	Platform	140	66	32	11	14	Vertical X-brace Cracked; to be Removed	Yes
	ST 0151	PROD-1	Platform	137	62	32	11	14	2 Vertical X-braces Damaged; 1 Replaced, 1 Repaired	Yes
	ST 0152	E	Platform	137	60	32	11	14	Toppled	Yes
	ST 0177	B	Platform	142	65	35	11	14	Sverely Damaged; being Removed	Yes
	ST 0177	B AUX	Platform	142	65	35	11	14	Sverely Damaged; being Removed	Yes
	ST 0188	CA	Platform	143	81	38	11	5	Deck Severely Damaged; Deck Shimmed, not Grouted; Deck had Large Cantilever; being Replaced; Jacket Undamaged	Yes
KERR-MCGEE CORPORATION	SS 0230	A	Platform	120	62	42	8	18		No
MOBIL OIL EXPLORATION & PRODUCTION	SS 0072	A	Platform	29	48	6	6	8	Total Loss, Collapsed; all Wells Abandoned Deck Broke Off; Jacket Broke Off 2' Above Mudline	Yes
	PL 0009	#4	Caisson	40	80	6	8	80		Yes
	PL 0009	#9	Caisson	40	85	6	8	80	Toppled	Yes
	PL 0010	#17	Caisson	40	84	5	8	77	Leaning 12 Deg.	Yes
	PL 0010	#18	Caisson	35	87	7	8	77	Toppled	Yes
MURPHY E&P	PL 0020	#13	Platform		67					Yes
	SS 0093	#6	Platform		69					Yes
	SS 0114	HEADER	Platform	54	65	13	6	8	Toppled	Yes
	SS 0114	#41	Platform	41	63	13	6	8	Leaning 30 Deg.	Yes
	SS 0118	E	Platform	54	54	16	6	4	Toppled; Heavy Rainbow Slick 2 MI by 800 YDS	Yes
	SS 0119	#5	Platform	43	73	16	6	3	Leaning 10 Deg.	Yes
	ST 0086	C	Platform	92	76	21	8	24	Pipeline Junction Platform T-21 Toppled; Hit by MODU	Yes
	PL 0019	#14	Caisson	30	81	12	8	80	Leaning 25 Deg.	Yes
	PL 0019	#33	Caisson		81					Yes
	PL 0019	#48	Caisson		86					Yes
	PL 0020	#28/#29	Caisson		81					Yes
	PL 0020	#33	Caisson		85					Yes
	PL 0020	#40/#42	Caisson		86					Yes
	SS 0090	#7	Caisson		91					Yes
	SS 0093	#12	Caisson		77					Yes
	SS 0093	53	Caisson	32	87	10	7	10	Toppled	Yes
	SS0094	#23	Caisson	48	87	20	7	9	Leaning 5 Deg.	Yes
	SS 0112	#4	Caisson	48	86	20	6	4		Yes
	SS 0112	#5	Caisson	47	86	20	6	4	Leaning 6 Deg.	Yes
	SS 0112	#6	Caisson	48	87	19	6	4	Leaning 5 Deg.	Yes

Table 2-1 (b): Best Estimate of Steel Jacket Platforms and Caissons Damaged in Hurricane Andrew in OCS Waters

OPERATOR NAME	BLOCK	PLATFORM DESCRIPTION					MAX. SIG. WAVE HT. HURRICANE ANDREW (M)	DISTANCE OF PLATFORM FROM HURRICANE EYE (MILES)	PLATFORM DAMAGE DESCRIPTION	DAMAGE CONFIRMED BY OPERATOR
		NAME OR WELL NO.	TYPE	WATER DEPTH (FEET)	YEAR INSTALLED	DISTANCE FROM SHORE (MILES)				
MURPHY E&P	SS 0113	#37	Caisson	50	81	13	6	7		Yes
	SS 0113	#41	Caisson	41	85	13	6	7		Yes
	SS 0113	#45	Caisson	45	90	15	6	7	Toppled	Yes
	SS 0113	#47	Caisson	47	89	18	6	7	Leaning 48 Deg.	Yes
	SS 0113	#48	Caisson	47	89	18	6	7	Toppled	Yes
	SS 0113	#51	Caisson	47	92	13	6	7	Leaning 15 Deg.	Yes
	SS 0113	#52	Caisson	47	90	13	6	7	Leaning 5 Deg.	Yes
	SS 0113	N	Caisson	48	88	13	6	7		Yes
	SS 0114	#1	Caisson	54	63	13	6	8	Toppled	Yes
	SS 0114	#46	Caisson	50	83	18	6	8	Leaning 35 Deg.	Yes
	SS 0114	#50	Caisson	46	84	19	6	8	Toppled	Yes
	SS 0114	#51	Caisson	50	82	13	6	8	Leaning 30 Deg.	Yes
	SS 0114	#55	Caisson	40	81	13	6	8	Leaning 45 Deg.	Yes
	SS 0114	#57	Caisson	50	86	18	6	8	Toppled	Yes
	SS 0114	#60	Caisson	51	85	13	6	8	Toppled	Yes
	SS 0114	#61	Caisson	53	86	18	6	8	Leaning 10 Deg.	Yes
	SS 0117	#2	Caisson	50	81	18	6	6	Leaning 30 Deg.	Yes
	SS 0117	#3	Caisson	53	82	18	6	6	Leaning 20 Deg.	Yes
	SS 0117	#7	Caisson	55	90	15	6	6	Leaning 45 Deg.	Yes
	SS 0118	#9	Caisson	53	92	21	6	4	Toppled	Yes
	SS 0119	Q	Caisson		82	16	6	3		Yes
	SS 0120	#2	Caisson	35	83	18	6	1	Leaning 5 Deg.	Yes
	SS 0120	#4	Caisson	40	83	17	6	1	Leaning 15 Deg.	Yes
	SS 0134	#20	Caisson	60	85	20	7	1	Toppled	Yes
	SS 0134	#7	Caisson	60	83	20	7	1	Leaning 45 Deg., Structure Repaired	Yes
	SS 0135	#10	Caisson	113	83	20	7	1	Leaning 10 Deg., Structure Repaired	Yes
	SS 0135	#5	Caisson	45	82	22	7	1	Leaning 5 Deg.	Yes
	SS 0136	#1	Caisson	65	83	32	7	2	Leaning 30 Deg.	Yes
	SS 0136	#2	Caisson	70	87	32	7	2	Leaning 20 Deg.	Yes
	ST 0086	#20	Caisson	110	83	22	8	24		Yes
SAMEDAN OIL CORPORATION	ST 0172	A	Platform	107	64	37	10	1	Platform Leaning 20 Deg., 360 Deg. Tear NE Jacket Leg, 2 Deck Legs Sheared, Rest Bent; SE Pile Pullout 15'; Adjacent East Pile Pullout 5'	Yes
STONE PETROLEUM CORPORATION	SP 0023	CA	Platform	61	62	15	6	70		No
TRUNKLINE	ST 0072	T-21	Platform	61	69	18	8	12	Toppled	Yes
TRUNKLINE	ST 0052	T-23	Platform	63	69	18	8	12	Damaged	Yes
TRUNKLINE	SS 0139	T-25	Platform	62	69	18	8	12	Heavily Damaged	Yes
UNOCAL	SS 0215	C	Platform	110	62	37	8	14	Toppled, Pre-existing Damage Scheduled for Removal Prior to Andrew	Yes

Table 2-2: Summary of Damaged Steel Jacket Platforms

Gulf of Mexico Area	Year Installed	Water Depth (ft.)	Distance of Platform from Hurricane Eye (Miles)	Number of Platforms	Condition of Platforms
South Timbalier (ST)	1948 - 1969	up to 100'	12	2	5 Toppled 6 Extensively damaged- being removed 5 with minor to heavy damage 1 with deck severly damaged- being replaced
		101' - 150'	1 to 5	2	
			14 - 21	10	
	1970 - 1977	up to 100'	24	1	
	1978 - 1981	101' - 150'	5, 18	2	
Ship Shoal (SS)	1948 - 1969	up to 50'	8	2	4 Toppled 3 with moderate to heavy damage 2 with unknown damage
		51' - 100'	4 to 12	3	
		101' - 150'	14, 18	2	
	1970 - 1977	43'	3, 24	2	
South Pass (SP)	1962	61'	70	1	Unknown damage
South Pelto (PL)	1967	unknown	unknown	1	Unknown damage
			Total	28	

Table 2-3: Summary of Damaged Caissons

Gulf of Mexico Area	Year Installed	Water Depth (ft.)	Distance of Caisson from Hurricane Eye (Miles)	Number of Caissons	Condition of Caissons
South Timbalier (ST)	1983	110'	24	1	Unknown damage
Ship Shoal (SS)	1954 - 1969	54'	8	1	9 Toppled 20 Leaning up to 48 degree 7 with unknown damage
	1977	unknown	unknown	1	
	1981 - 1992	up to 50'	1 to 10	22	
		51' - 100'	1 to 8	9	
		101' - 150'	1	1	
		unknown	unknown	2	
South Pelto (PL)	1980 - 1987	up to 50'	77 to 80	5	2 Toppled 2 Leaning up to 25 degree 6 with unknown damage
		unknown	unknown	5	
			Total	47	

Table 2-4: General Trends - Damaged Steel Jacket Platforms and Caissons

Item	Units	Damaged Platforms and Caissons	Damaged Platforms Only
Average Water Depth	ft.	69.1	101.2
Average Age	years	17	30
Average Distance to Center Path of Storm	miles	14.6	13.6
Average Distance from Shore	miles	20.2	25.4
Average Hs	ft.	24.6	29.2
Oldest Damaged Structures (year installed)	-	SS072A (1948) * SS118E (1954) * ST130A (1958) *	SS072A (1948) SS118E (1954) ST130A (1958)
Newest Damaged Strutures (year installed)	-	SS0113#51 (1992) ** SS0118#9 (1992) ** SS090#7 (1991) **	ST134W (1981) ST188-CA (1981) ST086C (1976)

Notes : * Damaged Steel Jacket Platform
** Damaged Caisson

Table 2-5: Platform Failures in Hurricanes Prior to Andrew **

HURRICANE NAME	HURRICANE DATE	OPERATOR NAME	BLOCK	PLATFORM CHARACTERISTICS NAME	YEAR INSTALLED	WATER DEPTH (ft.)	NUMBER OF PILES
GRANDE ISLAND	9/1/48	HUMBLE	GRAND ISLAND	2	Unknown	50	Temp. w/ unbraced piles
		HUMBLE	GRAND ISLAND	1	Unknown	50	Temp. w/ unbraced piles
CARLA	9/1/61	PLACID	EUGENE ISLAND 198	Unknown	Unknown	102	2
		SHELL	EAST CAMERON UK	Unknown	Unknown	Unknown	4
		ZAPATA	VERMILLION 104	Unknown	1960	60	4
HILDA	10/3/64	CATC	EUGENE ISLAND 208	A	Unknown	100	8
		CATC	EUGENE ISLAND 208	C	1959	96	8
		CATC	EUGENE ISLAND 208	D	Unknown	97	8
		GULF	SHIP SHOAL 154	B	Unknown	60	6
		GULF	SHIP SHOAL 154	H	Unknown	60	6
		GULF	SHIP SHOAL 169	A	1961	60	4
		PLACID	EUGENE ISLAND 198	B	1961	102	2
		PURE	SHIP SHOAL 253	Unknown	1964	172	8
		SIGNAL	SHIP SHOAL 149	B	Unknown	50	8
		SINCLAIR	EUGENE ISLAND 175	A	1955	87	16
		SHELL	EUGENE ISLAND 188	Unknown	1958	70	4
		TENNECO	SHIP SHOAL 198	C	1959	96	8
		TENNECO	SHIP SHOAL 199	A	1959	101	8
		UNION	EUGENE ISLAND 276	Unknown	1964	172	8
BETSY	9/9/65	CATC	WEST DELTA 69	#1	Unknown	125	3
		CATC	WEST DELTA 70	#3	Unknown	125	3
		FORREST	WEST DELTA 97	Unknown	Unknown	167	4
		GULF	WEST DELTA 117	A	1962	205	8
		GULF	WEST DELTA 117	B	Unknown	215	8
		PHILLIPS	MAIN PASS 129	Unknown	Unknown	92	4
		PURE	WEST DELTA 118	Unknown	Unknown	192	4
		SHELL	SOUTH PASS 24	Unknown	Unknown	60	4
CAMILLE	8/17/69	GULF	SOUTH PASS 61	A	1968	280	8
		SHELL	SOUTH PASS 70	A	1969	310	16
		SHELL	SOUTH PASS 70	B	1969	327	16
CARMEN	8/7/74	ODECO	SHIP SHOAL 119	A	Unknown	51	36
		ODECO	SHIP SHOAL 119	F	Unknown	51	36
FREDERIC	8/1/79	ODECO	SOUTH PELTO 19	#4	Unknown	30	3
		ODECO	SOUTH PELTO 19	#11	Unknown	30	3
		ODECO	SOUTH PELTO 19	#13	Unknown	30	3
JUAN	10/27/85	ODECO	SOUTH PELTO 19	OBM	1961	30	4
		ODECO	SOUTH PELTO 19	SWP	Unknown	30	3
		ODECO	SOUTH TIMBALIER 86	A	1955	95	16

** Platform failures (toppled in storm or severely damaged and then removed) only.
Other platforms may have been damaged but not reported.
Caissons not included.

Table 2-6: Summary of Qualitative Case Studies

PLATFORM NAME	OPERATOR NAME	WATER DEPTH (Feet)	YEAR INSTALLED	SIG. WAVE HEIGHT (Meters)	DISTANCE FROM EYE (Miles)	DAMAGE DESCRIPTION	REASON SELECTED
EW 826 A	BRITISH PETROLEUM EXPLORATION	488	1988	12	11	SURVIVED	ONE OF THE DEEPEST WATER PLATFORMS EXPOSED TO LARGE WAVES IN ANDREW
MC 311 A	SHELL OFFSHORE, INC.	425	1978	11	32	GRATING AND PIPING DAMAGED OR GONE	PLATFORM IN AREA OF LARGEST WAVES
SS 72 A	MOBIL OIL EXPLORATION & PRODUCTION CO.	29	1948	6	8	TOTAL LOSS; COLLAPSED; ALL WELLS ABANDONED	ONE OF THE OLDEST GULF PLATFORMS
SS 215 C	UNOCAL	100	1962	8	14	TOPPLED; DECK SEPARATED AND FELL TO SEA FLOOR	KNOWN DAMAGE PRIOR TO ANDREW
ST 152 E	CHEVRON USA INC	137	1960	11	14	TOPPLED; LYING ON BOTTOM	PLATFORM HAD TRIPOD REINFORCEMENTS
ST 172 A	SAMEDAN OIL CORPORATION	107	1964	10	1	PLATFORM LEANING 20 DEG; 360 DEG. TEAR NE JACKET LEG; 2 DECK LEGS SHEARED-REST BENT; PILE PULLOUT	OBSERVED FOUNDATION FAILURE
ST 177 B AUX	CHEVRON USA INC	140	1965	11	6	SEVERELY DAMAGED; JOINT FAILURE	SIMILAR TO ST177B AND ST151K USED IN QUALITATIVE ANALYSIS
ST 188 CA	CHEVRON USA INC	143	1981	11	5	DECK TORN OFF AND FOUND 200' AWAY, DECK SHIMMED, NOT GROUTED; JACKET UNDAMAGED	DECK TORN OFF

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: South Timbalier 0172 - Platform A
Company Name	: Samedan
Damage Summary	: Damaged - to be salvaged

Platform Information

Platform Type	:
Water Depth (ft)	: 107
Number of Piles	: 6
Number of Wells	:
Installation Date	: 1964
Design Criteria	:
Deck Elevation	: Sub-cellar @ 29'-0", Cellar @ 36'-6", Prod @ 47'-0"

Comments

- Deck removed 9/92, jacket removed 2/93
- South East pile pulled out 15 feet
- Adjacent east pile pulled out 5 feet
- North East and adjacent east deck leg sheared below cellar deck
- Remaining deck legs are bent and/or partially collapsed
- 360 degree tear in NE jacket leg (-10')
- 360 degree tear and collapse of north X-brace
- Structure leaning 20 degrees
- Foundation failure apparent

Table 2-7 Qualitative Case Study - 1

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: Mississippi Canyon 311 - Platform A (Bourbon)
Operator Name	: Shell
Damage Summary	: Survived

Platform Information

Platform Type	:
Water Depth (ft)	: 425
Number of Piles	:
Number of Wells	:
Installation Date	: 1978
Design Criteria	: Unknown
Deck Elevation	: Lower deck elev. +51'-0" T.O.S.

Andrew Conditions

Hindcast Hs	: 11 Meters
Distance from eye	: 32 Miles

Comments

- 8 to 10 sheets of grating torn up on lower deck around conductors (elev +51'-0")
- 90% of PVC drain piping suspended from lower deck gone (elev. +47'-0")
- Cage around ladder from +51' down to oil and water sump flattened right below lower deck at elevation +47'
- Sump landing grating missing and steel pipe knocked at bottom of oil and water sump severed
- Grating missing at +12' elevation
- Damage to underdeck piping indicates very large waves

Table 2-8 Qualitative Case Study - 2

Section 3
Capacity Assessments

3.1 APPROACH

The capacity assessments consisted of explicit nonlinear structural analysis to more thoroughly investigate individual platform performance in Andrew. Platform loads and resistance were based primarily upon the API RP 2A 20th edition. The overall intent was to provide input for use in the calibration process described in Section 4. The specific platforms selected for assessment were based upon a vote by all participants during the earlier stages of the project. Thirteen platforms were evaluated including those that survived, were damaged or collapsed during Andrew.

3.2 STRUCTURAL ANALYSIS

Two types of structural analysis were performed: a static pushover to determine the platform capacity for use in the calibration, and a typical design level code check (per API RP 2A) for use by API TG 92-5.

The static pushover is the typical approach used by the industry to determine the maximum lateral hydrodynamic load carrying capacity of offshore platforms. This load can then be traced back to a specific wave height that can cause platform failure. The static pushover is a somewhat simplified analysis technique since it does not consider mass, damping and rate-of-loading effects associated with wave loading. However, recent comparisons of the static pushover with a more complex dynamic analysis indicates that the static pushover provides a good estimate of platform capacity [PMB, 1993].

The static pushover consists of a representative "snapshot" of lateral wave forces acting on the platform, including any wave forces acting on the deck, and then applying the forces in a step-wise increasing manner until the platform collapses. The corresponding base shear acting on the platform at time of failure is used to define the platform capacity. Special nonlinear computer elements are used to mimic the nonlinear behavior of the jacket braces, legs, piles and soils. Further descriptions of the static pushover can be found in several references [Bea, et al., 1988; Lloyd and Clawson, 1983; Titus and Banon, 1988].

The design level code check was performed to provide information to API TG 92-5 for testing of several assessment approaches being studied by the TG. This analysis indicates the base shear at which the first brace, joint, pile or pile-soil platform components reach a unity check of 1.0 per API RP 2A requirements, safety factors included. The analysis also determined the global base shear acting on the platform per the API RP 2A 20th edition wave load recipe.

PMB performed structural analysis on nine of the platforms using the PMB computer code CAP (Capacity Analysis Program). The remaining four platforms were analyzed by the platform owners with results provided to PMB for inclusion in the project. Exxon analyzed two platforms using INTRA (Karma), Chevron analyzed one platform using CAP, and Amoco analyzed one platform using USFOS.

3.3 LOAD AND RESISTANCE RECIPE

The load and resistance recipe used for the capacity analysis was based primarily upon the API RP 20th edition, with several modifications as required for this project. Appendix G provides details on the recipe used for the project. Some of the more debatable issues, such as choice of Fy (steel yield strength) or the joint capacity equation, were based upon a vote by participants.

Key parameters of the recipe were as follows:

- **Factors of safety.** The recipe eliminates factors of safety in order to compute an unbiased platform capacity. This is necessary to calibrate analysis results with observed behavior.

- **Material strength.** Most of the platforms were fabricated using steel with a 36 ksi nominal yield strength. Participants voted on using a yield strength of 42 ksi for these cases to account for the increase from nominal to mean and to account for increased strength due to strain rate effects (rapid loading in storms) [Chen and Ross, 1977]. Material strength based upon mill certificates was used where available.

- **Brace capacity (buckling).** The brace capacity is defined by equation D.2.2-2 of API RP 2A LRFD [API, 1989].

- **Effective length (K) factors.** The effective length K factor of "K", "diagonal" and "X" bracing schemes was taken as 0.65, based upon results of laboratory tests [Grenda et al., 1988] and analytical studies [Earl and Teer, 1989]. The length was taken as node-to-node of the computer model (not face-to-face of the leg). For X bracing, the member length is taken as one-half the node-to-node length (i.e. out-of-plane buckling is not considered due to the compensating effect of the tension brace).

- **Ungrouted joints.** API RP 2A equations for joint capacity without safety factors. These equations represent a lower bound for joint capacity. The "mean" capacity for K-joints (not used here) would be approximately 15 to 30 percent higher.

- **Grouted joints.** The same API RP 2A equations as for ungrouted joints, but use an equivalent thickness for the leg based upon the strength contributions from the leg, grout and pile [UEG, 1983].

- **Lateral soil capacity.** The AIM projects and other assessment-type studies have typically used degraded soil-pile capacity to develop p-y nonlinear soil springs for pushover analysis. This is based upon the assumption that the soil strength is degraded at the time of the peak wave due to cyclic action of other large waves during storm build-up. However, recent laboratory test by Exxon [Hamilton, 1992], indicate that for pushover type analysis, the static lateral soil strength is a better measure. Therefore, lateral p-y soil strength (as defined by API RP 2A) was used for all of the analysis. Vertical t-z springs used static soil strength (no degradation).

- **Wave loads on the deck.** In cases where waves impact the deck, use the simplified procedure developed by API Task Group 92-5, shown in Table 3-1.

3.4 ANALYSIS RESULTS — SUMMARY

Results are presented in two formats. The first is a summary of all the platform configurations and pushover analysis results necessary for calibration as shown in Tables 3-2 (survivals), 3-3 (damages) and 3-4 (failures). Further discussion of these tables is provided below.

The second format is complete details for each platform. These details can be found in Appendix B, and include a more complete description of the platform and further information on the computer models and analysis results. Further discussion of these detailed results using one of the platforms as an example is provided in Section 3.5.

The information found in each column of tables 3-2 to 3-4 is as follows:

- **Background Information** - the first few columns describe the general characteristic of the platforms.

- **Pushover Direction** – this is the direction from which a pushover analysis was performed for the particular platform – generally end-on, diagonal, broadside, or some combination. Additional directions were performed in some cases as required for calibration.

- **Expected Maximum Hindcast Base Shear** – an estimate of the hindcast maximum global base shear during Andrew based on the maximum hourly hindcast wave height in that direction. The wave height and associated current are taken directly from the Oceanweather Andrew hindcast. These values are taken from detailed evaluations performed in Section 4 and hindcast base shear tables given for individual platforms in Appendix B.

- **Base Shear at First Component Failure** – the global base shear at which the indicated platform component (brace, joint, pile, soils) "fails" based upon the load and resistance recipe described in Section 3.3.2. This information contains no factors of safety. These are important for some cases in the calibration process.

- **Base Shear at Ultimate Platform Capacity** – the global base shear at which the platform is considered as collapsed based upon the static pushover analysis. This is the key parameter for the calibration process.

- **Ratio of Ultimate Capacity to Andrew Base Shear (BS)** – provides an approximate estimate of reserve in platform capacity compared to Andrew base shear estimate.

- **System Factor** – provides an estimate of platform capacity remaining after first member failure, computed as the ratio of the load level at platform collapse to the load level at first member failure.

- **Collapse Mode** – the collapse mode for the platform based upon results of the static pushover. Frame failure indicates failure of jacket structural members, which could occur due to a combination of multiple failure of K-joints or braces, or first yield/ hinge formation in the leg(s). Pile hinge failure indicates double hinge formation in multiple piles or single hinge formation in a number of piles with associated large displacement at deck level. Pile pullout failure indicates soil failure due to axial loads. Displacement (deck) greater than 4' to 5' indicates that the platform capacity has been achieved due to large deck displacements and the analysis has been terminated.

These tables provided results for single or multiple directions for the 13 platforms analyzed. The ultimate capacity of the five, 4-legged (4-pile) platforms installed from 1964-1981 in water depths up to 170 ft. ranges from 1265 kips to 2006 kips. For these platforms, the minimum ratio of ultimate capacity to Andrew base shear varies from 0.79 to 1.84 and the system factor varies from 1.13 to 2.23. Frame failure consisting of multiple K-joint failures dominated the collapse mode for three of these platforms and pile failures for the other two. One platform indicated double hinging of piles and other platform indicated pile pullout as their collapse modes.

The ultimate capacity of MC397 platform with 4 main legs and 4 skirt legs installed in 1991 in 468 ft. water depth is in range of 11,566 kips to 13,718 kips. The high ratio of ultimate capacity to Andrew base shear indicates redundant structural framing. Soil failure leading to pile pullout was established as the failure mode for this platform.

Six 8-legged platforms evaluated in this project were installed between 1958 to 1965 in water depths up to 184 ft. and one 8-legged platform was installed in 1978 in 343 ft. water depth. The ultimate capacity of the pre-1969 platforms is estimated to range between 2,450 kips to 4,426 kips. The estimate of minimum ratio of ultimate capacity to Andrew base shear varied from 0.73 to 1.61 and the corresponding system factors varied from 1.1 to 1.7. A combination of frame and pile member failures lead to formation of collapse mechanism for these platforms. The frame member failures for all of these platforms were due to failure of multiple K-joints.

The minimum ultimate capacity of an 8-legged MC311 platform with 8 main piles and 8 skirt piles, installed in 343 ft. water depth is estimated as 17,900 kips. The ratio of ultimate capacity, corresponding to frame failure, to the Andrew base shear was 2.8.

3.5 ANALYSIS RESULTS — DETAILS (Example Platform)

An explanation of the detailed quantitative information provided for each platform follows, using ST151 K, which survived Andrew, as an example.

Figure 3-1 shows an overall view of the platform which is an 8 legged (external legs double battered) platform installed in 1963 in a water depth of 137 ft. The pile-legs are grouted and the platform has K-joints in the broadside loading direction and diagonally braced in the end-on loading direction. Figures 3-2 and 3-3 show the typical member sizes. The platform survived Andrew with no damage.

Figure 3-4 shows the nonlinear computer model of the platform used for the static pushover analysis. The model consisted of a fully coupled nonlinear jacket-foundation system. The force deformation relationship used to model each of the primary platform elements is shown. The model included the following:

- **Deck** – typical linear beam-column elements since no inelastic response is anticipated. The deck framing was simplified for the analysis.
- **Legs, Piles and Conductors** – nonlinear beam-column elements which carry both bending and axial loads.

- **Braces** – buckling-type struts for braces which are weaker than the joint (i.e. diagonals in the end-on loading direction) and nonlinear elastic-plastic truss elements for the braces which are stronger than the joints (i.e. the broadside loading direction K-joints). For this later case of "weak" K-joints, the elastic-plastic modeling of the K-joint failure was based upon discussion with participants involved in a series of confidential laboratory joint tests. However, as discussed later at the project meetings, depending upon the characteristics of the steel material, elastic-plastic modeling may not be correct for all joint failures. Post-Andrew inspections of platform 177B indicate that the K-joints were completely sheared at the chord, which would be more properly modeled with strut-type modeling that portrays load shedding. Modeling of this type of failure mode obviously needs further investigation. The struts and truss elements carry axial loads only.

- **Soils** – Nonlinear p-y, t-z and q-z springs.

Once developed, the model was used for static pushover analysis in order to determine the platform capacity for use in the calibration process. CAP actually uses a "pseudo-static" procedure for static pushover. This procedure is an equivalent dynamic analysis with special dynamic control parameters that allows the platform to incur significant deflections while still remaining stable. This method is used since it is more robust (i.e. ability to achieve a solution), requires less user interaction and is less time consuming than a conventional static pushover. The pseudo static pushover determines only the platform's ultimate capacity, and not the post peak capacity available with a conventional static pushover. Since establishing the platform capacity is the primary goal of pushover analyses, the pseudo static approach was adequate for most of the work of this project. Comparisons of the static and pseudo-static analyses run for other projects and in-house PMB studies have shown that the capacities computed by the two methods are similar.

Figure 3-5 shows the deformed platform shape (deflections have been amplified for better visual effect) for intermediate results of the broadside pushover analysis at a lateral load level (load step 13) where several K-joints in two bays have failed. Figure 3-6 shows the nonlinear events and deformed platform shape just prior to collapse, with several K-joints failed and double hinging beginning to occur in the piles. Figures 3-7 and 3-8 show the nonlinear events and deflected platform shape for diagonal direction pushover analysis.

Figure 3-9 shows the force-deflection plot for the broadside pushover analysis. Deflections were taken at the lower deck elevation. At the initial application of loads, the platform responds in a slightly nonlinear manner due to the nonlinear foundation system (nonlinearity in the pile-soils). The first K-joint fails at an applied load of 2,330 kips. The platform then begins to respond in an increased nonlinear manner due to inelasticity in the jacket. Initial pile yield occurs at an applied lateral load level of 3,250 kips. It is followed by yielding of all eight piles and large displacement at deck level. The pushover load level of 3,500 kips is taken as the capacity of the platform in this direction.

Figure 3-10 shows results of the static pushover in the diagonal direction. For waves approaching the diagonal direction, the analysis indicated that the first nonlinear event occurs due to first yield of a pile section at a pushover load level of 3,005 kips followed by failure of a K-joint at 3,350 kips in the second bay of the jacket and successive failure of other pile sections. The pushover load level of 3,500 kips is taken as the capacity of the platform in this direction.

3.6 API BASED EVALUATION

Following the pushover analysis, additional analyses were performed to determine the design level code checks for use by API TG 92-5. Due to time constraints, these values were only determined for some of the platforms. The design level code check was performed to provide information to API TG 92-5 for testing of several assessment approaches being studied by the Task Group. For some of the platforms, the checks were performed by the platform owner. The following three items were established:

- **API 100-Year Base Shear** – the global base shear acting on the platform in each direction based on the API RP 2A 20th edition wind/wave/current load recipe.

- **API Design Level Check** – the global base shear at which the indicated platform component (brace, joint, pile, soils) first fails API RP 2A 20th edition (API, 1993) design requirements (including all factors of safety).

- **Reserve Strength Ratio (RSR)** – The ratio of ultimate capacity of a platform to the API 100-year base shear. These values should be considered as a rough approximation of RSR as the ultimate capacity values used are dependent upon the wave kinematics and forces due to Andrew seastates, and the load and resistance recipe followed in this project.

The results of this evaluation performed on 10 platforms are summarized in Tables 3-5 to 3-7. In addition, the ultimate capacity analysis and Andrew base shear estimates are also included for comparison purposes.

The results presented in these tables indicate that for the 8-legged platforms, the API base shear estimate for diagonal and broadside directions is approximately 30 percent to 50 percent higher than that for the end-on direction. The reasons for this are platform orientation with true North, and reduced seastate parameters, especially current magnitude in the end-on direction. The API 100-year return period base shear estimate for these platforms varies from 1,630 kips to 4,164 kips for the end-on direction and from 2,314 kips to 5,357 kips for the broadside and diagonal directions. This resulted in RSRs in the end-on directions to range between 1.40 to 1.72 compared to RSRs of 0.88 to 1.30 for the broadside and diagonal directions for four 8-legged platforms. For platform WD90A, estimates of RSR are low – 0.58 and 0.78 for the diagonal and end-on directions, respectively.

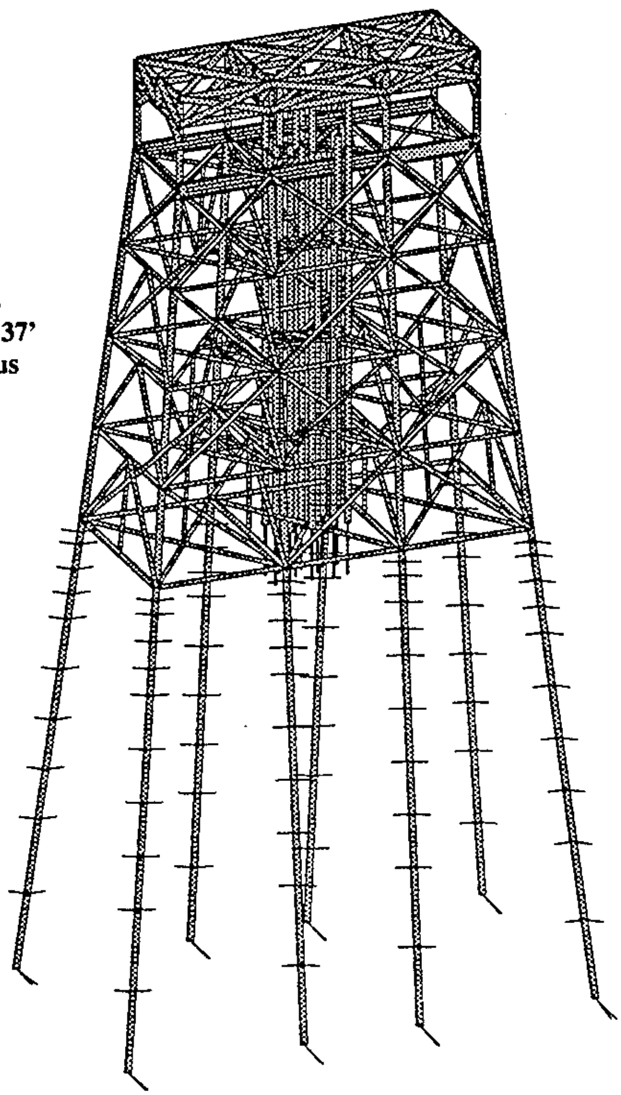
The ratios of ultimate capacity to the minimum API design level capacity ranges from 2.4 to 6.4 for all the 8-legged platforms. These high values are because these platforms were installed before 1966, when design criteria were significantly lower than present standards (e.g., 25 vs. 100-yr conditions). Therefore, the first member failure per current API loads occurs at a low base shear. The ultimate capacity value is high since it excludes all factors of safety and follows the recipe discussed in Section 3.3

The API 20th edition 100-year base shear for the five 4-legged platforms varies from 878 kips to 1365 kips. The minimum RSR of these platforms ranges between 0.99 to 2.28 and the ratio of ultimate capacity to minimum API design level capacity varies from 1.63 to 3.14.

The minimum ratios of ultimate capacity to Andrew base shear for all platforms except WD90A are lower than corresponding values for RSR, which indicates that Andrew load effects were higher than per the API 20th edition. Platform WD90A is located away from the Andrew path, and thus its ratio of ultimate capacity to Andrew base shear is high (1.6 to 1.69) and is the likely reason for its survival.

Platform ST151K

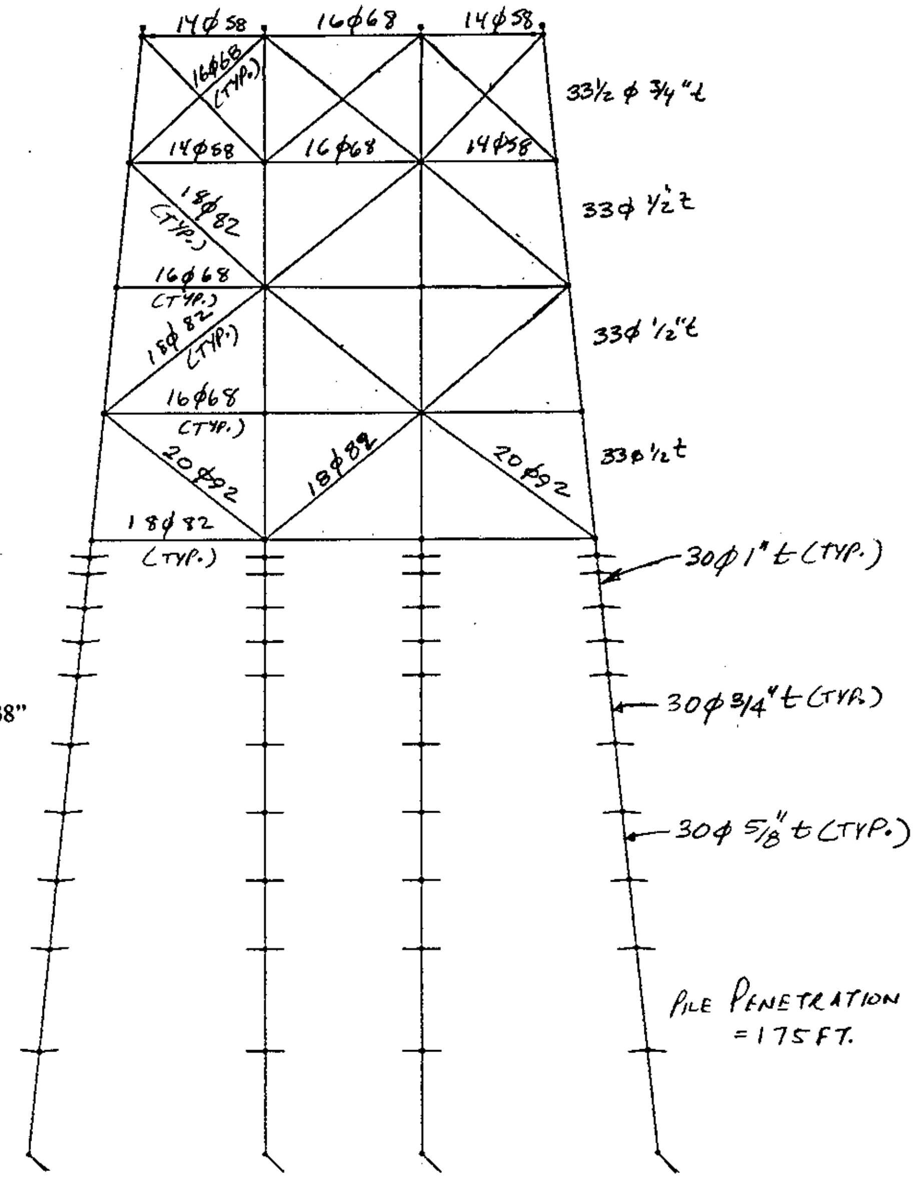
- Survived Andrew
- Year Installed - 1963
- Water depth = 137'
- 12 - 30" Dia. Conductors
- Cellar Deck Elevation = 37'
- Grouted Leg-Pile Annulus



CAP Z X Chevron ST151K - Analytical Model

Project: ChevST151K Model: pushxy Version: 1

Figure 3-1 Background Information - Platform ST151K

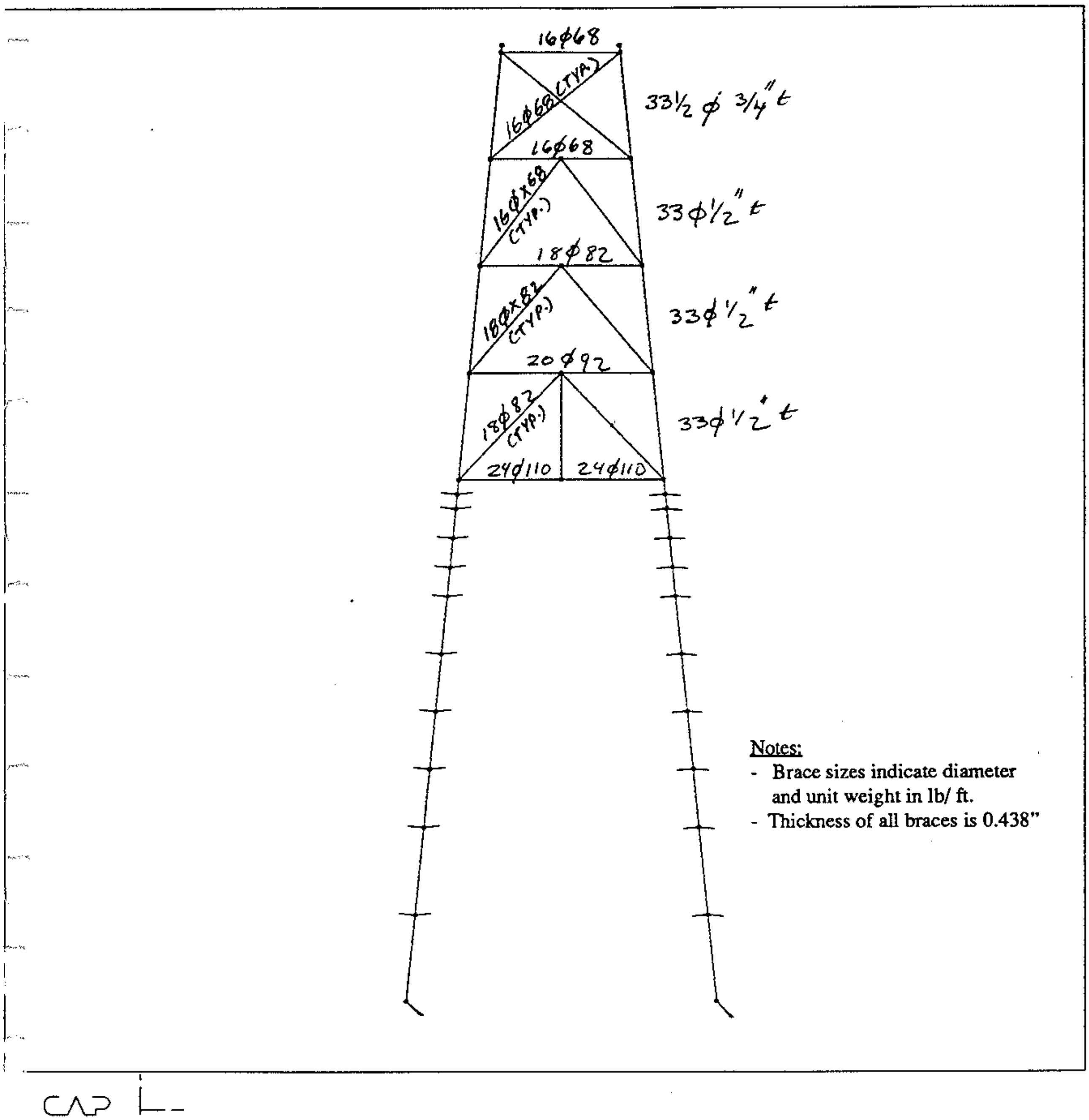


Notes:
- Brace sizes indicate diameter and unit weight in lb/ ft.
- Thickness of all braces is 0.438"

CAP Z X

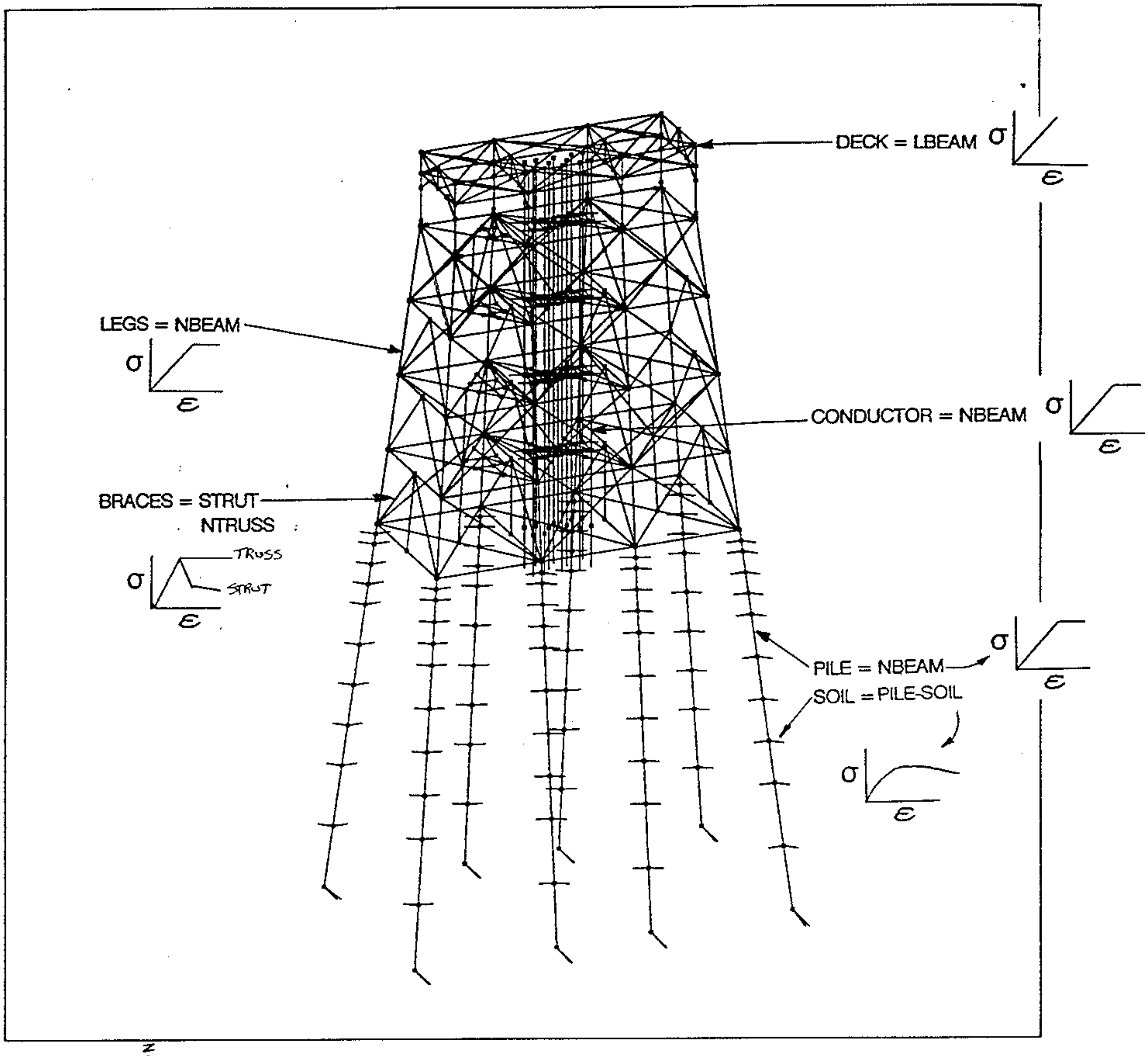
Project: ChevST151K Model: pushx Version: 1

Figure 3-2   Structural Configuration: Rows A & B - ST151K



Project: ChevST151K Model: pushx Version: 1

Figure 3-3 Structural Configuration: Rows 1 to 4 - ST151K



CAP

Chevron ST151K Pushover Model - Simplified

Project: ChevST151K Model: pushy Version: 1

Figure 3-4 Nonlinear Analysis Computer Model - ST151K

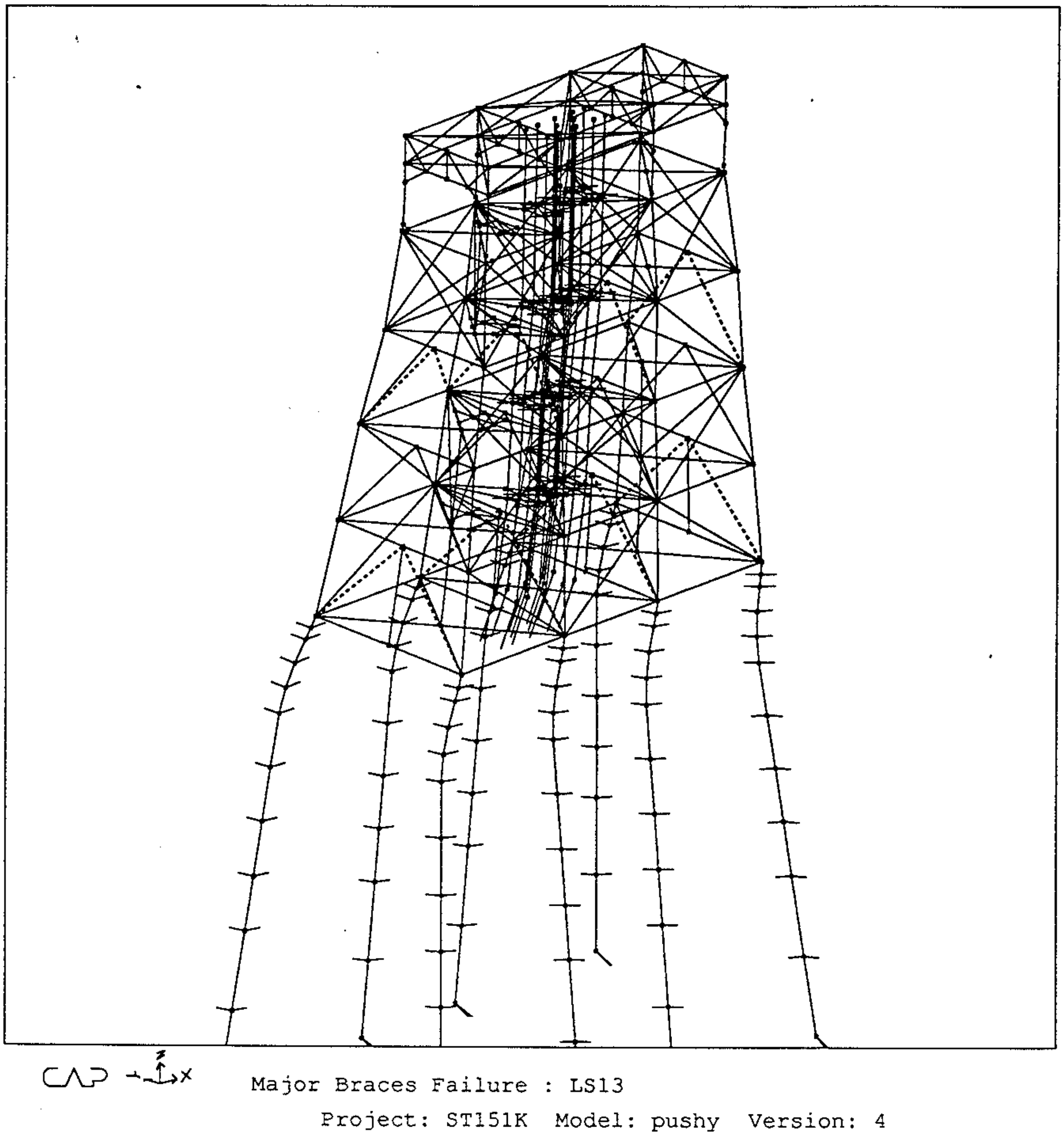


Figure 3-5 Pushover Analysis Joint/ Brace Failures: Broadside Direction - ST151K

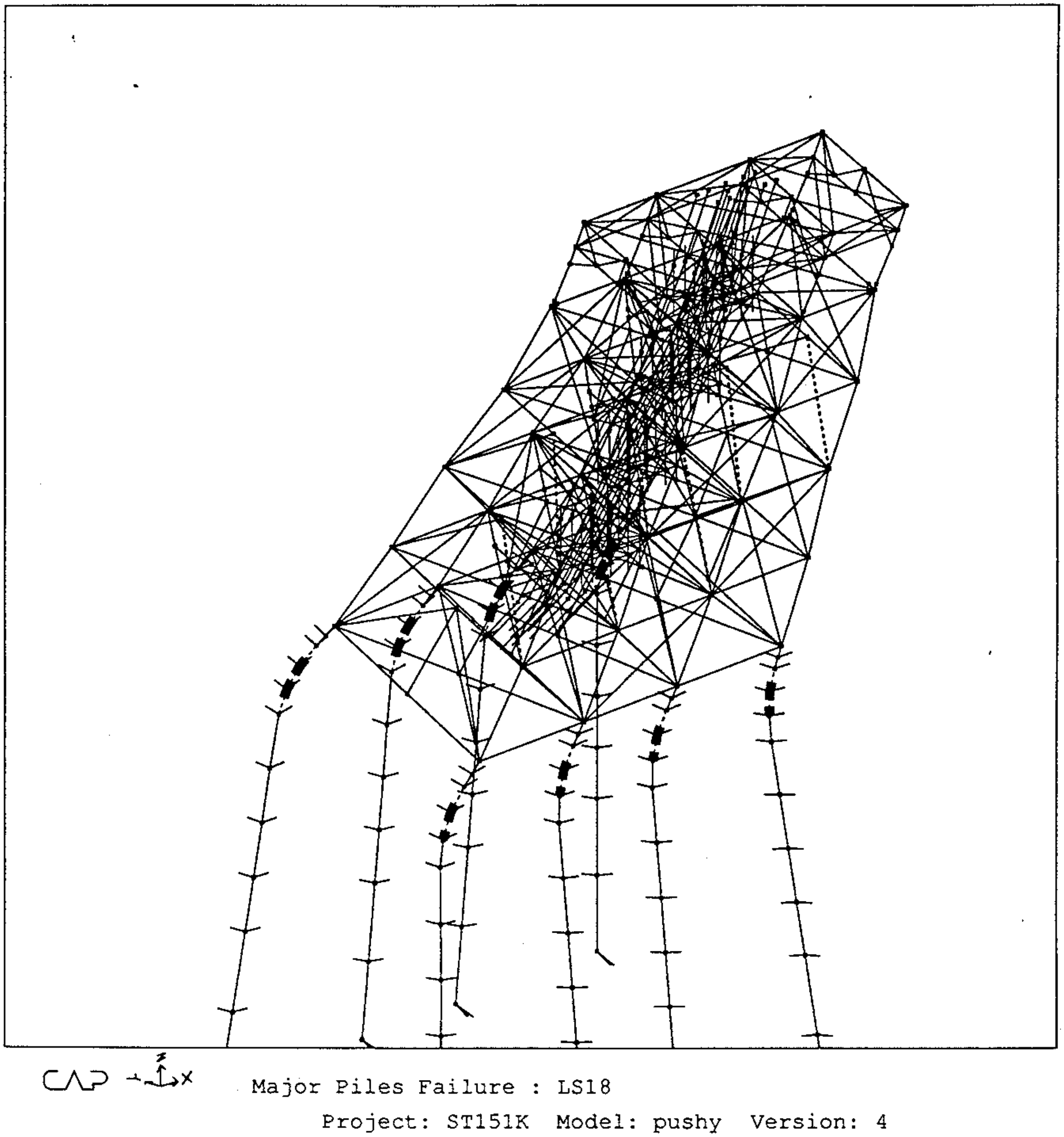


Figure 3-6 Pushover Analysis Ultimate Collapse Results: Broadside Direction - ST151K

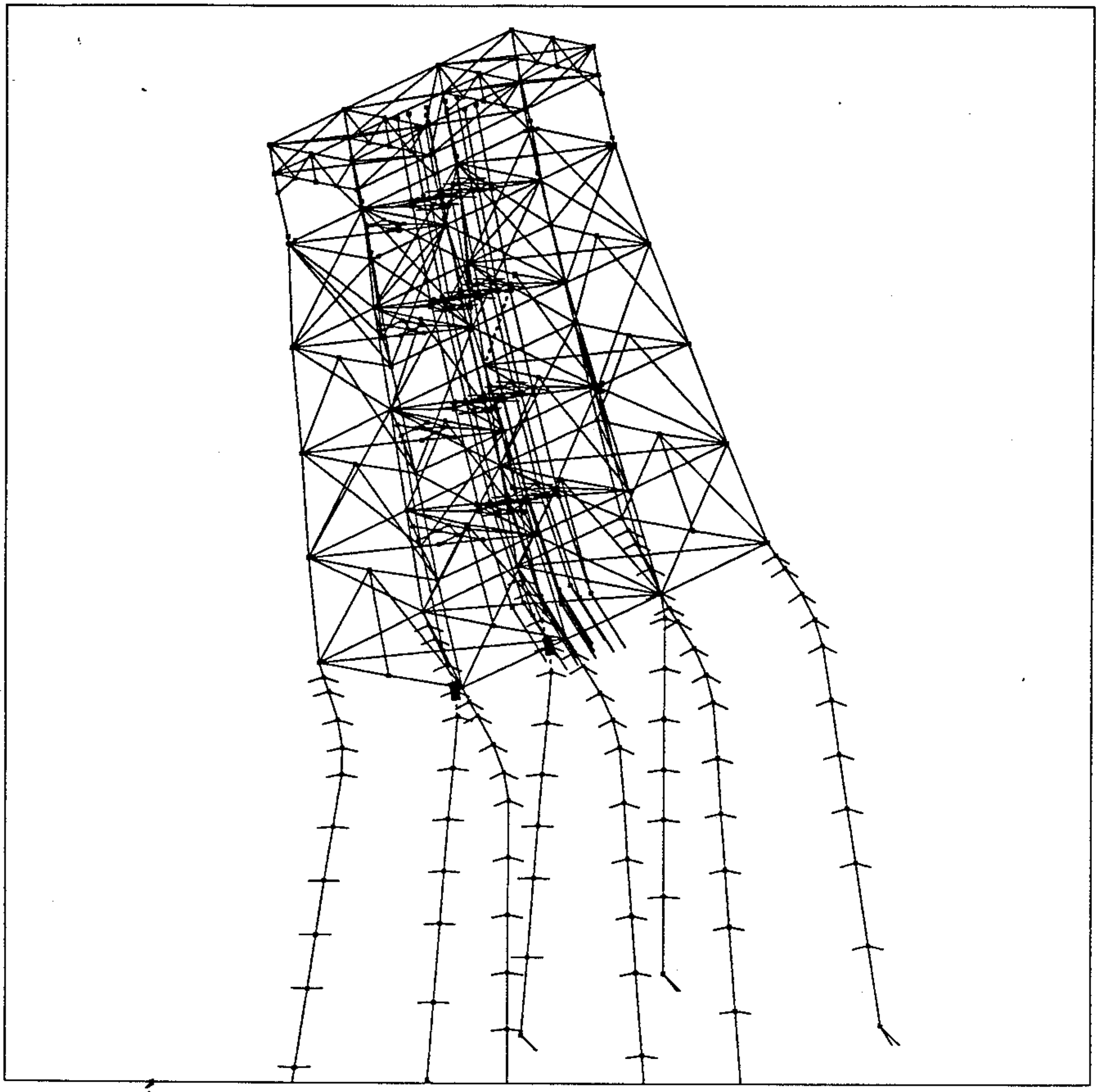


Figure 3-7 Pushover Analysis First Member Failures: Diagonal Direction - ST151K

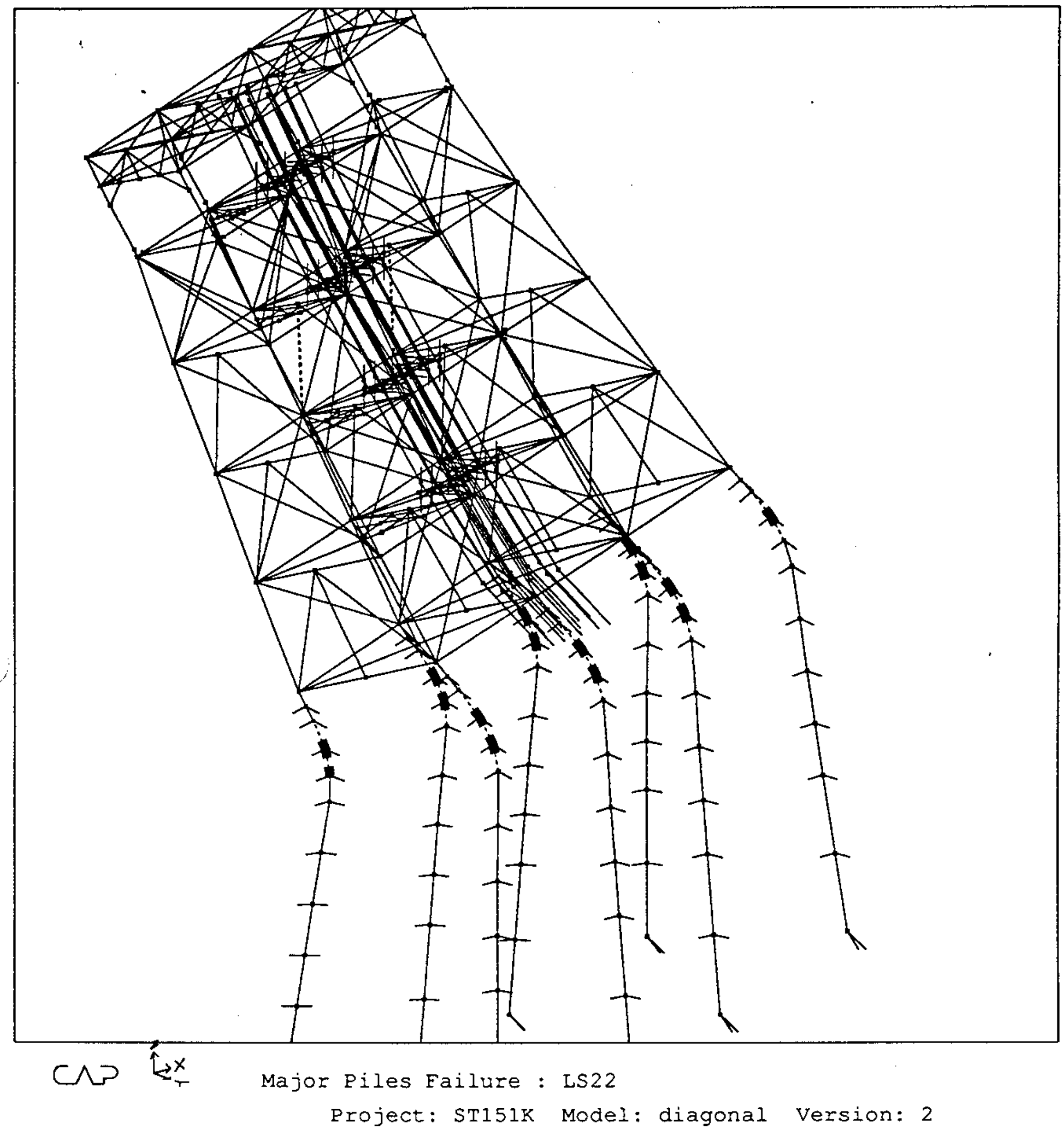
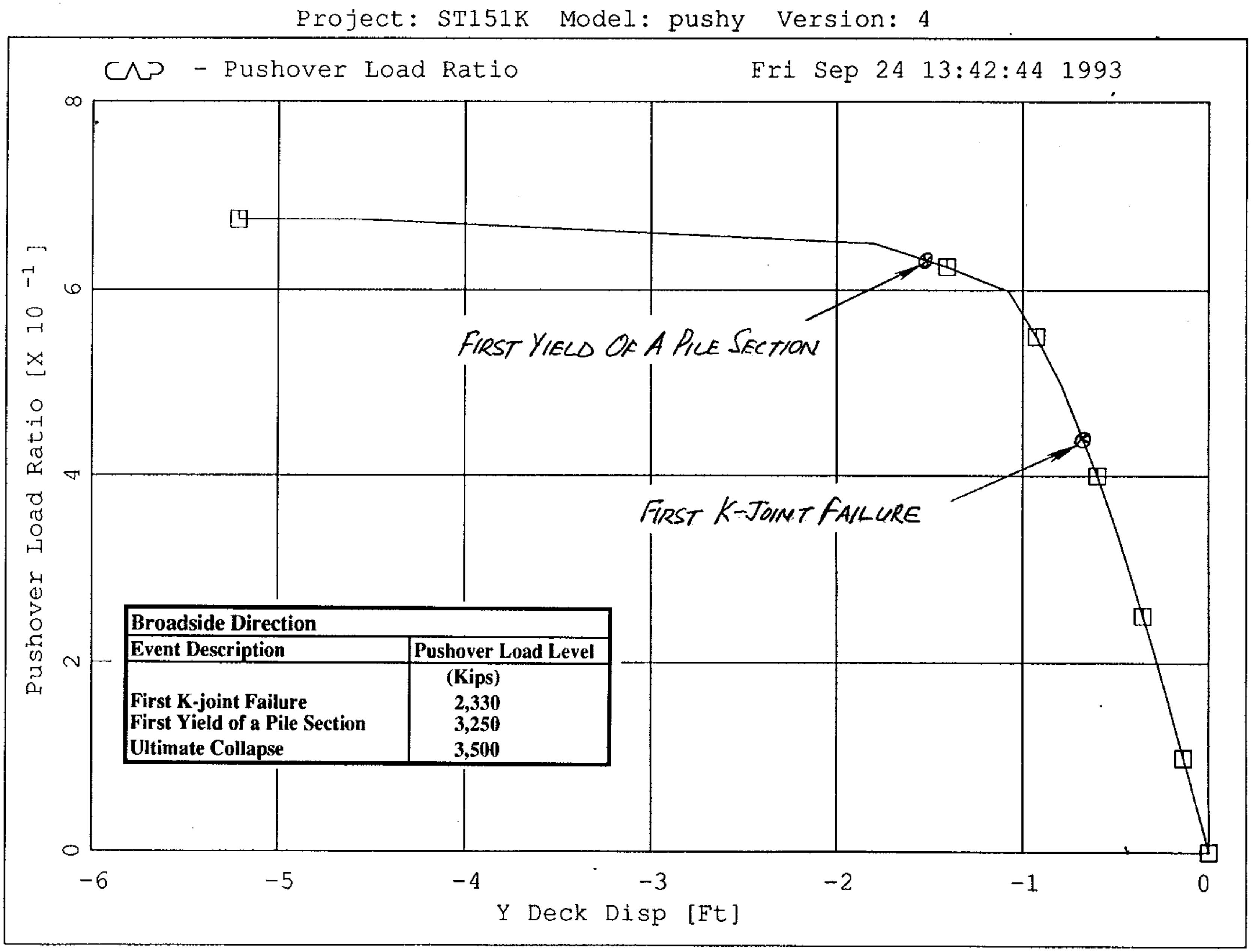
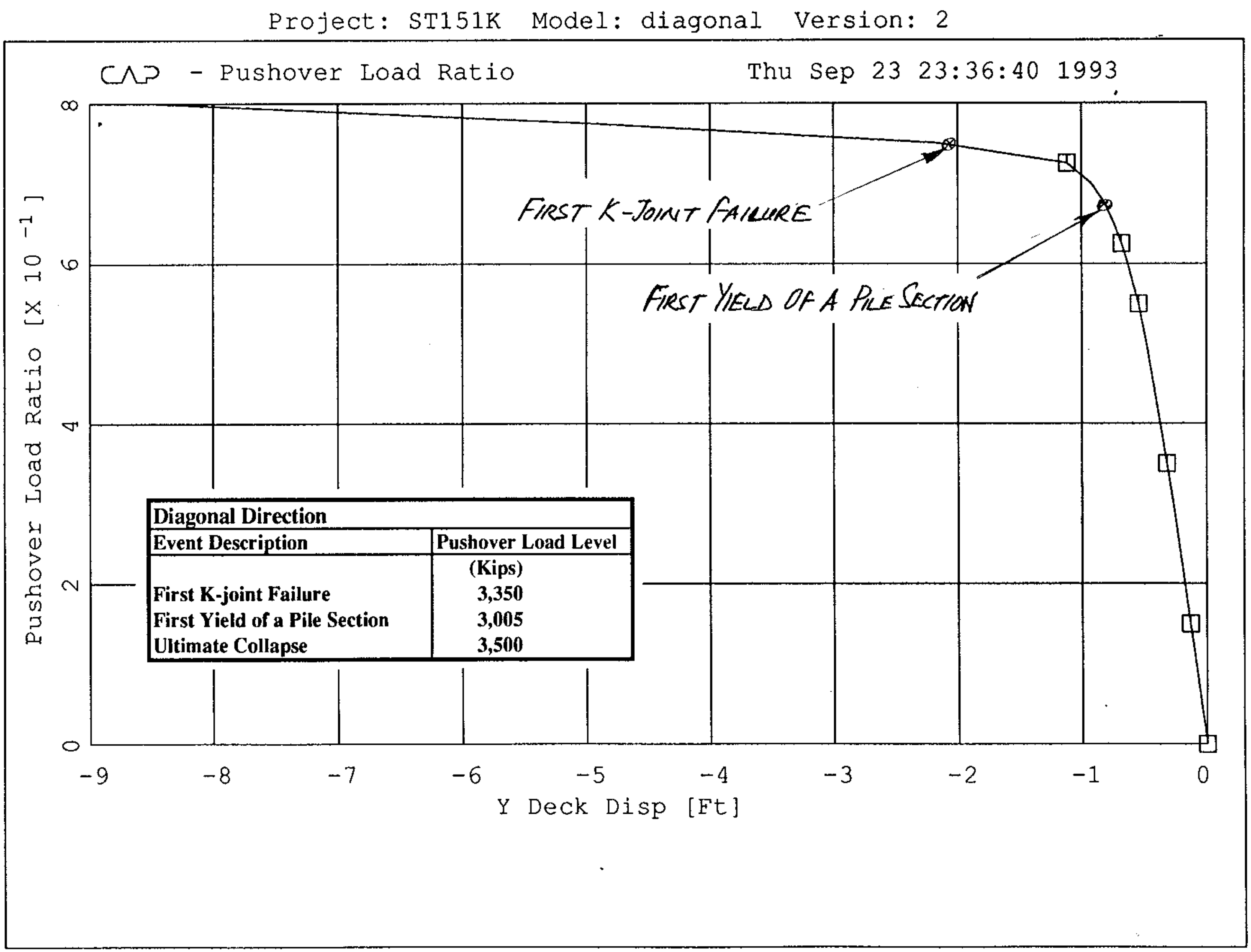


Figure 3-8 Pushover Analysis Ultimate Collapse Results: Diagonal Direction - ST151K



Broadside Direction	
Event Description	Pushover Load Level
	(Kips)
First K-joint Failure	2,330
First Yield of a Pile Section	3,250
Ultimate Collapse	3,500

Figure 3-9 Pushover Analysis Results: Broadside Direction - ST151K



Diagonal Direction	
Event Description	Pushover Load Level (Kips)
First K-joint Failure	3,350
First Yield of a Pile Section	3,005
Ultimate Collapse	3,500

Figure 3-10 Pushover Analysis Results: Diagonal Direction - ST151K

Preliminary Deck Force Guidelines

The purpose of these guidelines is to provide a simple yet conservative method for predicting the wave loads on fixed platform decks for use in the "Task Group on Assessment of Existing Platforms to Demonstrate Fitness for Purpose." The method is presented as an interim method for the purpose of evaluating the three-tiered screening procedure for assessing fixed offshore platforms. The procedure returns the maximum wave-induced deck load, which is assumed to occur at the same time as the maximum base shear, and the moment induced by the deck load. The steps for computing the base shear and overturning moment caused by the deck follows. This procedure is expected to return a relatively conservative deck force and moment, with a COV on the order of 35%.

1. Given the crest height (see note below), compute the wetted "silhouette" deck area, A, projected in the wave direction, θ_w. The silhouette area is defined as the shaded area in Figure 1. The area, A, is computed as:

   $A = A_x \cos\theta_w + A_y \sin\theta_w$,

   where θ_w, A_x and A_y are as defined in Figure 2.

   For lightly framed "sub-cellar" deck sections with no equipment, such as a "scaffold" deck comprised of angle iron, use one-half of the silhouette area. The areas of the deck legs and bracing above the cellar deck are part of the silhouette area. Deck legs and bracing members below the bottom of the cellar deck should be computed along with jacket members in the jacket force calculation procedure.

2. Use Stream Function Wave Theory or equivalent with specified wave period, water depth, and current speed to compute the maximum horizontal fluid velocity, V, at the crest elevation or the top of the deck structure, whichever is lower. A directional spreading factor of 0.88 is applied to the velocity.

3. The wave force on the deck, F, is computed as follows:

   $F = 1/2\, \rho\, C_D\, (V + \alpha U)^2\, A$

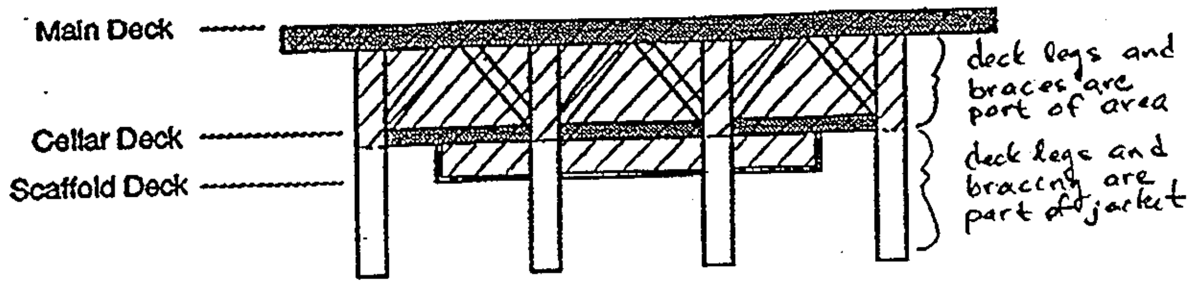
   where U is the current velocity, with the same blockage factor, α, as specified for the jacket. The drag coefficient, C_D, is given in the table below:

Deck Type	end-on and broadside	oblique (45-degrees)
modern deck (very dense)	2.5	1.9
heavily equipped "older" deck	2.0	1.5
bare "older" deck	1.6	1.2

4. The overturning moment on the jacket due to wave loads on the deck is obtained by applying the deck load to a point 60% of the way between the lowest point of the silhouette area and the lower of the wave crest or top of deck.

note: The above procedure relies on the use of an adjusted wave height when using Stream Function Wave Theory (or equivalent) in order to return crest elevations that are in closer agreement with measured data. The wave heights for the deck force calculation should be 1.056 times the wave heights used for the jacket force calculation.

Table 3-1 API Preliminary Deck Force Guidelines (April 30, 1993)



Elevation View of Platform Deck

Figure 1: Silhouette Area Definition.

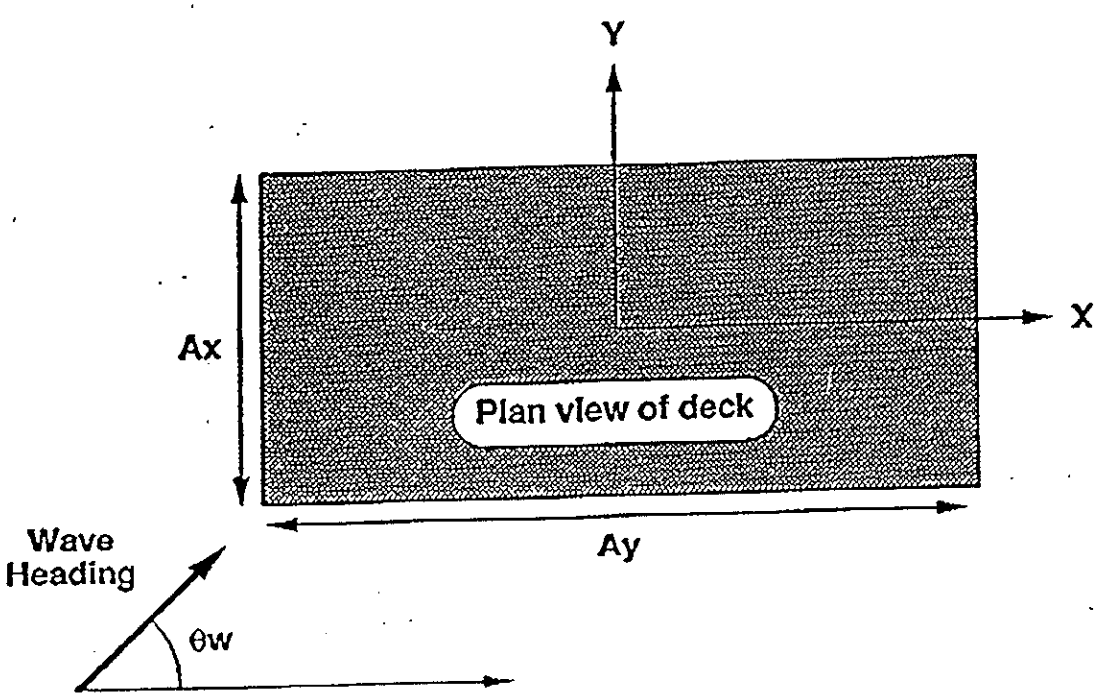


Figure 2: Wave heading and Direction Convention.

Table 3-1 API Preliminary Deck Force Guidelines (April 30, 1993) (Continued)

Table 3-2: Capacity Analysis Results for Survival Platform Cases

Platform	Platform Characteristics: Configuration	Water Depth (Feet)	Year Installed	Pushover Direction	Expected Maximum Hindcast Base Shear (kips)	Base Shear (Kips) from Static Pushover Analysis at First Component Failure: Brace	Joint	Pile	Soils	at Ultimate Capacity	Ratio Ult. Cap./ Andrew BS	System Factor = Ult. Cap./ BS at First member failure	Platform Collapse Mode
ST 151 K	8 Leg - Grouted Double Battered K Braced	137	1963	Broadside	4473	-	2330	3250	-	3500	0.78	1.50	Frame Failure/ Pile Hinge
				Diagonal	4765	-	3350	3005	-	3500	0.73	1.16	Pile Hinge
				End On	1230	3556	-	3734	-	3900	3.17	1.10	Pile Hinge
ST130Q	4 Leg - Grouted Double Battered K Braced	170	1964	Diagonal	1214	-	-	1118	1265	1265	1.04	1.13	Pile Pullout/ Plunging
ST 134 W	4 Leg - Ungrouted 3 Sides Battered Diagonal Brace	137	1981	End On	1307	-	-	1620	-	1923	1.47	1.19	Pile Hinge
				Diagonal	1118	-	-	1632	-	1915	1.71	1.17	Pile Hinge
WD 90 A	8 Leg - Grouted Double Battered K Braced	184	1964	Diagonal	1856	3130	2393	-	3130	3130	1.69	1.31	Pile Pullout
				End On	2029	3104	2614	-	3267	3267	1.61	1.25	Pile Pullout
MC 311	8 Leg - Ungrouted Double Battered Diagonal & X Brace	343	1978	Broadside	5606	14000	-	-	-	20700	3.69	1.48	Frame Failure/ Pile Hinge
				Diagonal	6382	-	-	-	-	17900	2.80	-	Frame Failure
MC 397	4 Leg - Ungrouted Double Battered X Braced	468	1991	Broadside	4938	-	-	-	-	13718	2.78	-	Pile Pullout
				Diagonal	3144	-	-	-	-	11566	3.68	-	Pile Pullout

Table 3-3: Capacity Analysis Results for Damage Platform Cases

Platform	Platform Characteristics			Pushover Direction	Expected Maximum Hindcast Base Shear (kips)	Base Shear (Kips) from Static Pushover Analysis at First Component Failure				Base Shear (Kips) from Static Pushover Analysis	Ratio	System Factor =	Platform Collapse Mode
	Configuration	Water Depth (Feet)	Year Installed			Brace	Joint	Pile	Soils	at Ultimate Capacity	Ult. Cap./ Andrew BS	Ult. Cap./ BS at First member failure	
ST 52 (T-23)	4 Leg - Grouted Double Battered K Braced	63	1969	Diagonal 2 (67.5 Deg)	1092	-	901	1686	-	**2006**	**1.84**	**2.23**	**Frame Failure/ Pile Hinge**
SS 139 (T-25)	4 Leg - Grouted Double Battered K Braced	62	1969	Orthogonal	1691	1006	671	1174	-	**1342**	**0.79**	**2.00**	**Frame Failure/ Displacement > 5'**
ST 161 A	8 Leg - Grouted Double Battered Diag. & K Braced	118	1964	Broadside	3973	-	3670	-	-	**4426**	**1.11**	**1.21**	**Frame Failures/ Displacement > 4'**

Table 3-4: Capacity Analysis Results for Collapsed or Severely Damaged Platform Cases

Platform	Platform Characteristics			Pushover Direction	Expected Maximum Hindcast Base Shear (kips)	Base Shear (Kips) from Static Pushover Analysis					Ratio	System Factor	Platform Collapse Mode
						at First Component Failure							
	Configuration	Water Depth (Feet)	Year Installed			Brace	Joint	Pile	Soils	at Ultimate Capacity	Ult. Cap./ Andrew BS	Ult. Cap./ BS at First member failure	
ST 177 B	8 Leg - Grouted Double Battered K Braced	142	1965	Diagonal 1 (22.5 Deg)	5150	-	2452	3677	-	4168	0.81	1.70	Frame Failure/ Pile Hinge
				Diagonal 2 (45 Deg)	3938	-	3226	3917	-	4378	1.11	1.36	Frame Failure/ Pile Hinge
				End On	2394	3517	-	3341	-	4220	1.76	1.26	Frame Failure/ Pile Hinge
ST 151 H	8 Leg - Grouted Single Battered K Braced	137	1964	Broadside	2552	-	2494	4156	-	4156	1.63	1.67	Frame Failure/ Pile Hinge
				Diagonal	4206	-	2666	3999	-	3999	0.95	1.50	Frame Failure/ Pile Hinge
				End On	1160	2464	-	3450	-	3450	2.97	1.40	Frame Failure/ Pile Hinge
ST 130 A	8 Leg - Grouted Double Battered K Braced	180	1958	Broadside	1858	-	1450	1550	-	2450	1.32	1.69	Frame Failure/ Pile Hinge
				Diagonal	2779	-	1300	1800	-	3000	1.08	2.31	Frame Failure/ Pile Hinge
				End On	1582	1800	-	1550	-	2800	1.77	1.81	Frame Failure
ST 72 (T-21	4 Leg - Grouted Double Battered K braced	61	1969	Orthogonal	1615	1438	1050	1633	-	1984	1.23	1.89	Frame Failure/ Displacement >5'

Table 3-5: API Based Evaluation and Comparison with Andrew Results for Survival Platform Cases

Platform	Platform Characteristics: Configuration	Water Depth (Feet)	Year Installed	Pushover Direction	API 100 yr Base Shear (kips)	API Design Level Check Base Shear (Kips) at First Component Failure: Brace	Joint	Pile	Soils	Pushover Analysis Ultimate Capacity	Ratio of Load at First member Failure/ API 100-year Base Shear: Brace	Joint	Pile	Soils	Approx. Andrew Hindcast Max. Base Shear	RSR = Ult. Cap./ API 100 yr BS	Ratio of Ult. Cap./ API design level capacity	Ratio of Ult. Cap./ Andrew BS	Platform Collapse Mode
ST 151 K	8 Leg - Grouted Double Battered K Braced	137	1963	Broadside	3959	2236	1011	2360	2287	3500	0.56	0.26	0.60	0.58	4473	0.88	3.46	0.78	Frame Failure/ Pile Hinge
				Diagonal	3843	2293	1404	2050	1802	3500	0.60	0.37	0.53	0.47	4765	0.91	2.49	0.73	Pile Hinge
				End On	2565	1715	1134	2004	2399	3900	0.67	0.44	0.78	0.94	1230	1.52	3.44	3.17	Pile Hinge
ST130Q	4 Leg - Grouted Double Battered K Braced	170	1964	Diagonal	1217	403	742	487	790	1265	0.33	0.61	0.40	0.65	1214	1.04	3.14	1.04	Pile Pullout/ Plunging
ST 134 W	4 Leg - Ungrouted 3 Sides Battered Diagonal Brace	137	1981	End On	1176	1272	784	1736	1505	1923	1.08	0.67	1.48	1.28	1307	1.64	2.45	1.47	Pile Hinge
				Diagonal	906	1424	1063	1804	1213	1915	1.57	1.17	1.99	1.34	1118	2.11	1.80	1.71	Pile Hinge
WD 90 A	8 Leg - Grouted Double Battered K Braced	184	1964	Diagonal	5357	2196	1280	2379	1098	3130	0.41	0.24	0.44	0.20	1856	0.58	2.85	1.69	Pile Pullout
				End On	4164	2888	1925	2500	1347	3267	0.69	0.46	0.60	0.32	2029	0.78	2.43	1.61	Pile Pullout

Table 3-6: API Based Evaluation and Comparison with Andrew Results for Damaged Platform Cases

Platform	Platform Characteristics			Pushover Direction	API 100 yr Base Shear (kips)	API Design Level Check Base Shear (Kips) at First Component Failure				Pushover Analysis Ultimate Capacity	Ratio of Load at First member Failure/ API 100-year Base Shear				Approx. Andrew Hindcast Max. Base Shear	RSR = Ult. Cap./ API 100 yr BS	Ratio of Ult. Cap./ API design level capacity	Ratio Ult. Cap./ Andrew BS	Platform Collapse Mode
	Configuration	Water Depth (Feet)	Year Installed			Brace	Joint	Pile	Soils		Brace	Joint	Pile	Soils					
ST 52 (T-23)	4 Leg - Grouted Double Battered K Braced	63	1969	Diagonal (67.5 Deg.)	878	2080	817	2601	1337	2006	2.37	0.93	2.96	1.52	1092	2.28	2.46	1.84	Frame Failure/ Pile Hinge
SS 139 (T-25)	4 Leg - Grouted Double Battered K Braced	62	1969	Orthogonal	1355	1524	821	1524	2346	1342	1.12	0.61	1.12	1.73	1691	0.99	1.63	0.79	Frame Failure Displacement > 5'

Table 3-7: API Based Evaluation and Comparison with Andrew Results for Collapsed or Severely Damaged Platform cases

Platform	Platform Characteristics: Configuration	Water Depth (Feet)	Year Installed	Pushover Direction	API 100 yr Base Shear (kips)	API Design Level Check Base Shear (Kips) at First Component Failu: Brace	Joint	Pile	Soils	Pushover Analysis Ultimate Capacity	Ratio of Load at First member Failure/ API 100-year Base Shear: Brace	Joint	Pile	Soils	Approx. Andrew Hindcast Max. Base Shear	RSR = Ult. Cap./ API 100 yr BS	Ratio of Ult. Cap./ API design level capacity	Ratio Ult. Cap./ Andrew BS	Platform Collapse Mode
ST 177 B	8 Leg - Grouted Double Battered K Braced	142	1965	Diagonal 1 (22.5 Deg)	4200	-	-	-	-	4168	-	-	-	-	5150	0.99	-	0.81	Frame Failure/ Pile Hinge
				Diagonal 2 (45 Deg)	4574	-	-	-	-	4378	-	-	-	-	3938	0.96	-	1.11	Frame Failure/ Pile Hinge
				End On	3011	-	-	-	-	4220	-	-	-	-	2394	1.40	-	1.76	Frame Failure/ Pile Hinge
ST 151 H	8 Leg - Grouted Single Battered K Braced	137	1964	Broadside	3614	2146	967	1968	2048	4156	0.59	0.27	0.54	0.57	2552	1.15	4.30	1.63	Frame Failure/ Pile Hinge
				Diagonal	3422	1601	1251	1564	1580	3999	0.47	0.37	0.46	0.46	4206	1.17	3.20	0.95	Frame Failure/ Pile Hinge
				End On	2368	1047	895	1619	2332	3450	0.44	0.38	0.68	0.98	1160	1.46	3.85	2.97	Frame Failure/ Pile Hinge
ST 130 A	8 Leg - Grouted Double Battered K Braced	180	1958	Broadside	2494	1415	381	1297	1301	2450	0.57	0.15	0.52	0.52	1858	0.98	6.43	1.32	Frame Failure/ Pile Hinge
				Diagonal	2314	1113	492	1426	1606	3000	0.48	0.21	0.62	0.69	2779	1.30	6.10	1.08	Frame Failure/ Pile Hinge
				End On	1630	814	784	1583	2268	2800	0.50	0.48	0.97	1.39	1582	1.72	3.57	1.77	Frame Failure
ST 72 (T-2	4 Leg - Grouted Double Battered K braced	61	1969	Orthogonal	1365	1495	854	2456	2136	1984	1.10	0.63	1.80	1.56	1615	1.45	2.32	1.23	Frame Failure/ Displacement >5'

Section 4
Calibration

4.1 APPROACH

The calibration process involves a comparison of analytically predicted platform performance to observed platform performance. The end result is a bias factor that can be used to improve the estimate of platform safety.

The first step is to identify the analytical theory to be used for the calibration. Traditional "Bayesian" updating techniques were followed and applied to the situation for Andrew (single large storm). The detailed analytical procedure was identified and a computer code was developed to perform the required complex calculations.

The key input for the process included results of the static pushover analysis, namely, the platform capacity and, in the case of damaged platforms, the pushover load level to cause observed damages. The other key input was metocean conditions at the specific platform site during Andrew, that were based upon the Oceanweather hindcast. The process was performed for each of the 13 selected platforms, and then the results combined to determine the final bias factor. Sensitivity studies were performed to evaluate the importance of various parameters and different groups of platforms.

The following sections provide some background discussion on calibration, followed by theoretical details of the analytical approach, a discussion of the step-by-step calibration procedure and finally a summary of the calibration results.

4.2 BACKGROUND ON CALIBRATION

More accurate assessments of risk or safety indices (betas) can be derived by using field experiences to modify or update the distribution parameters of model uncertainties. This process, sometimes called "Bayesian" analysis, was previously used in the Amoco-organized cooperative project on offshore platform reliability. Further discussions of Bayesian applications were given in the API PRAC Project 89-22 Report [Moses, 1991], and other literature [Tang, 1981; Marshall and Bea, 1976].

The general approach for Bayesian analysis is illustrated in Figures 4-1 and 4-2 [Petrauskas, 1992]. Figure 4-1 shows the primary components required for calibration, and how the experience of Hurricane Andrew provides an opportunity to calibrate existing force and resistance models used for platform assessment.

Figure 4-2 shows the "prior" load and resistance distributions that exist before the calibration. Variables that describe modeling uncertainties may be assigned to the mean load, the mean resistance, or both. The quantification of the Bayesian analysis requires assignment of an initial distribution (called the "prior" distribution) to the model uncertainty.

Predictions of safety of the structural system, conditional upon realization of specific values of the model variables, is done by conventional load-resistance reliability assessments. (Failure occurs when load effect exceeds resistance. Uncertainties not related to the modeling remain part of this reliability index calculation.)

These calculations lead through the Bayes theorem to a new and updated distribution (called the "posterior" distribution) of the model variables.

Based upon platform experience in Hurricane Andrew, the Bayesian calculations would shift their prior distributions to posterior distributions. If the platforms are found to "survive" when the prior distributions indicate failure, then the load and resistance distributions move further apart (survival data result), indicating the assessment process is more conservative than initially suspected. If the platforms are found to "fail" when the prior distributions indicate a survival, then the load and resistance distributions move closer together (failure data result), indicating the assessment process is less conservative than initially suspected.

These updated "posterior" variables, based upon actual experiences, can then be used as the basis for either code calibration or development of guidelines for reassessing structures. Sensitivity studies are needed to show that initial prior assumptions of the model variables have little effect on the assessments of reliability which follow the Bayesian analysis. Such conclusions were developed from parametric analysis in both the Amoco [Moses, 1976] and the recent PRAC 89-22 studies [Moses, 1991].

4.3 CALIBRATION APPROACH USED IN THE PROJECT

The last section provided general discussion of calibration and described some possible results. This section describes the specific analytical calibration approach used for the project.

4.3.1 The Bias Factor

The first task of the project was to determine what items or parameters would actually be calibrated. Several possibilities exist, for example, specific detailed portions of the load and resistance recipes, such as drag coefficients for use in the Morrison equation, or the "overall" load or resistance recipe itself.

While calibration of specific details of the recipe is attractive, it also requires substantial specific detailed information (i.e. Hurricane Andrew observations) from which to perform the calibration. It became apparent early in the project that this level of detailed observations would not be available. Calibration of the overall load or resistance recipe is also attractive, but it became apparent that the project could not accurately determine if the calibration should focus on the load or resistance portion of the recipe. In addition, such calibrations would require a greater in-depth effort beyond the scope of this project.

Therefore, it was decided to de-couple the calibration from either specific details of the recipe or even from load or resistance, and instead to introduce a bias factor of "B" that effects the "safety margin" of the platform, defined as the ratio of resistance (R) to load (S), or:

$$(R/S)\ \text{true} = B\ (R/S)\ \text{computed} \tag{4-1}$$

Thus, the "true" margin R/S equals the "computed" margin R/S (as per the assessment process) times a bias factor B to account for the inaccuracy in modeling. A value of B greater than 1.0 indicates (on average) conservatism in the assessment process. A value of B less than 1.0 indicates an (on average) unconservative process.

The bias factor B is what was "calibrated" by this project. The initial or "prior" value of B for the project was taken as 1.0, which assumes no bias in the existing assessment approach, and then updated via the calibration process to a new or "posterior" value based upon analytical versus observed results of platform survivals, failures and damage in Andrew. In actuality (as discussed later), this project has used a probabilistic approach for calibration where B is defined by a normal distribution type with a mean value of 1.0 and an uncertainty, defined as the coefficient of variation, of 30 percent for the prior.

4.3.2 Theory for Development of Bias Factor

The standard formula for calculating the probability of failure of a platform under wave loading provides the initial basis for the calibration approach. The formula reflects the variability in wave loads and capacities. The formula is then modified to introduce the additional uncertainty resulting from the current limits to a precise understanding about waves, wave forces, and the ultimate capacity of jacket systems. This modification allows a quantitative description of the expansion to the current state of information resulting from the inclusion of the Andrew data.

For clarity of presentation, attention here will be focused on the single most critical direction (an octant of the compass) of waves relative to the structure; this may be end on, broadside, diagonal or at (45°) to the structure's axis. In fact, the results have shown that this is an accurate assumption. Results are presented in Section 4.5 and sensitivity analysis results presented in Appendix C for the effect of an increase in the number of seastates justify the decision here to concentrate on a single direction.

The conventional formula for calculating the probability of failure of a structure is

$$P_f = \int_o^\infty \{1 - F_S(x)\} f_R(x)\, dx \tag{4-2}$$

where

f_R is capacity PDF (probability density function)

F_S is CDF (cumulative distribution function) of load

In general the random load, S, is, for example, the maximum load in any one-year period, and this case will be used below. The updating will also include the case in which S is the maximum load on the structure during Andrew.

The load is represented by the base shear, BS, which will be represented by

$$BS = C1\,[h + C2\,u]^{C3} \cdot \epsilon_o \tag{4-3}$$

in which h is a wave height and u is a current, while $C1$, $C2$ and $C3$ are coefficients specific to a particular platform and wave/current direction set (found by fitting this empirical equation to calculated base shears for various pairs of h and u values). Finally, ϵ_o is a lognormal random variable with median 1.0 and specified coefficient of variation (COV) (or, correspondingly, log standard deviation, $\sigma_{\ln\epsilon}$). ϵ_o represents wave-to-wave variability in the actual base shear given waves of the same h (and period, T_p) with the same current u. It is sometimes referred to as the variability in accurately predicting hydrodynamic forces (e.g., variabilities in the drag coefficient C_d).

Eq. 4-3 represents the (random) base shear associated with a specified wave height h and current velocity u (with specified directions). In any given 1 hour seastate with significant wave height h_s and current u, there will be a sequence of N waves with random wave heights, H. We assume N is approximately $3600/T_p$, where T_p is the period associated with the peak of the wave spectrum. It is assumed that the probability distribution of each H is the empirical Forristall distribution:

$$f_{H|H_s}(h \mid H_{s_j} = h_s) = \frac{\alpha 4^\alpha}{\beta H_s}\left(\frac{x}{H_s}\right)^{\alpha-1} \exp\left[-\frac{4^\alpha}{\beta}\left(\frac{x}{H_s}\right)^\alpha\right] \qquad (4\text{-}4)$$

in which $\alpha = 2.126$ and $\beta = 8.42$. (This implies that the mean of H given $H_S = h_s$ is 0.603 h_s.)

Under these assumptions, the CDF of the maximum base shear in one hour (given $H_s = h_s$ and the (random) current U has value u) is

$$\left\{\int F_{BS}(x \mid H=h,\, U=u_j) \cdot f_{H|H_s}(h \mid H_{s_j} = h_{sj})\, dh\right\}^{N_j} \qquad (4\text{-}5)$$

F_{BS} is the lognormal cumulative distribution implied by Eq. 4-3. Extending this approach to multiple hours with given values h_{s_1}, h_{s_2}, etc. and common current u, the (conditional) CDF of the maximum base shear is obtained:

$$\prod_{\substack{Hour\\ j}}^{No.ofSign.Hours} \left\{\int F_{BS}(x \mid H=h,\, U_j=u) \cdot f_{H|H_s}(h \mid H_{s_j} = h_{sj})\, dh\right\}^{N_j} \qquad (4\text{-}6)$$

There is randomness in the H_s level and the current U. This is modeled by assuming that there is a (deterministic) storm "profile," i.e., evolution of the significant wave height versus time, given by values $\hat{H}_{s_1}$, $\hat{H}_{s_2}$, etc. Furthermore, the uncertainty in the actual significant wave heights is represented by assuming that the true value in each hour is $H_{s_1} = \hat{H}_{s_1} \cdot \epsilon_1$, $H_{s_2} = \hat{H}_{s_2} \cdot \epsilon_1$, etc., where the random variable, ϵ_1, is lognormal with median 1 and $\sigma_{\ln\epsilon_1}$ and is assumed common from hour to hour. This model can be applied to represent either the maximum storm next year or the hindcast estimate of last year's Andrew event at a given site, except that the numerical values assigned to the $\hat{H}_{s_j}$

and the $\sigma_{\ln\epsilon_1}$ would change. The same model is used for the current U, with ϵ_2 representing the uncertainty (future or past) in the current. The final (marginal) CDF for the maximum base shear, F_{MBS}, during the multi-hour (unidirectional) "storm" is

$$F_{MBS}(x) = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} \prod_{\substack{Hour \\ j}}^{No.\,of Sign.\,Hours} \left\{ \int F_{BS}(x \mid H=h, U=u_j) \cdot f_{H|H_s}(h \mid H_{s_j} = h_{sj})\, dh \right\}^{N_j} f_{\epsilon_{H_s}}(\epsilon_1) f_{\epsilon_U}(\epsilon_2) d\epsilon_1 d\epsilon_2 \tag{4-7}$$

in which $f_{\epsilon_{H_s}}(\epsilon_1)$ and $f_{\epsilon_{1_j}}(\epsilon_2)$ are the PDF's of the "errors," respectively, in the hindcast of the significant wave heights and currents during Andrew (or in the "next year" case, the year-to-year variability in the maximum annual significant wave height about its median and the (associated) current about its median). Note that in Eq. 4-7, $h_{s_j} = \hat{H}_{s_j} \cdot \epsilon_1$ and $u_j = \hat{U}_j \cdot \epsilon_2$.

The CDF, F_{MBS}, of the maximum base shear is obtained by numerical integration given, in the Andrew case, the hindcast estimates of the significant wave height and current in each hour (while the waves approach from the critical octant), plus the structure specific base shear coefficients C_1, C_2, C_3, and finally a set of COV's (or approximately the $\sigma_{\ln\epsilon}$). With this, probability of failure can be calculated by numerical integration of Eq. 4-2, assuming a lognormal distribution on R, with a specified median and COV.

As discussed in Section 4.3.1, in order to correct for possible bias in the load and resistance recipe, a factor B has been introduced into the standard models used for wave loads and structural capacities. Failure therefore is presumed to be associated with $BR/S < 1$ rather than $R/S < 1$.

$$P_f(b) = \int_o^\infty \{1 - F_s(bx)\} f_R(x)\,dx \qquad (4\text{-}8)$$

This is easily calculated for a specified value of B using virtually the process as used above for P_f.

Eq. 4-8 is used in two ways. The first is to calculate the probability of failure next year. Because information about the value of B is imperfect, the assumption is made that it can be represented as a probabilistic variable, with *PDF:* $f_B(b)$; the dispersion (standard deviation) of this distribution represents what is referred to variously as "model or parameter uncertainty," "epistemic uncertainty," or "Type II uncertainty." The central value, B, is anticipated to be somewhere around unity.

Given the distribution B, the probability of failure is found as:

$$P_f = \int_o^\infty P_f(b)\, f_B(b)\,db \qquad (4\text{-}9)$$

in which $P_f(b)$ is given by Eq. 4-8.

Finally, the objective of updating is to modify the distribution on B in a manner consistent with the information/behavior observed during, for example, Andrew. The updating is based on Bayes theorem of probability theory which states:

$$f_B''(b) \propto f_B'(b) \cdot lk\left(b \mid \begin{matrix} new \\ information \end{matrix}\right) \qquad (4\text{-}10)$$

in which $f_B'(b)$ is the "prior" distribution of bias factor, B and $f_B''(b)$ is the "posterior" (i.e., before and after obtaining the new information); $lk\left(b \mid \begin{matrix} new \\ information \end{matrix}\right)$ is the "likelihood function" on b which reflects the information about b contained in the new observations.

For a specific structure, assume that a "success" is observed, i.e., that the capacity exceeded the load, then

$$\begin{aligned} lk(b\,|\,success) &= P[success\,|\,b] \\ &= 1 - P_f(b) \end{aligned} \tag{4-11}$$

where $P_f(b)$ was given in Eq. 4-8. Similarly, a failure implies

$$lk(b\,|\,failure) = P_f(b) \tag{4-12}$$

Instead of simple success or failure, assume that a degree of damage can be observed that can be interpreted as the occurrence within a particular range of the load to capacity ratio. For example, assume a pushover analysis suggests that n_1 braces fail at load/capacity $= \alpha_1$ and n_2 ($>n_1$) at α_2, then damage of n_1 but not n_2 braces implies the event that

$$\frac{1}{\alpha_2} \le b\frac{R}{S} \le \frac{1}{\alpha_1} \tag{4-13}$$

and

$$\begin{aligned} lk(b\,|\,damage) &= P\left[\frac{1}{\alpha_2} \le \frac{bR}{S} \le \frac{1}{\alpha_1}\,|\,b\right] \\ &= P\left[\frac{bR}{S} \le \frac{1}{\alpha_1}\,|\,b\right] - P\left[\frac{bR}{S} \le \frac{1}{\alpha_2}\,|\,b\right] \\ &= P_f(\alpha_1 b) - P_f(\alpha_2 b) \end{aligned} \tag{4-14}$$

As discussed in the following section, the above procedure was incorporated into a work station based computer code in order to simplify the required computations.

4.3.3 PF and C1C2C3 Computer Programs

The project developed two computer codes to perform the computations. The programs are called PF, which is used to develop the likelihood curves, and C1C2C3 which is used to find the hydrodynamic force coefficients for each platform. The programs are written in Fortran and operate on a SUN workstation. Each is described briefly below.

PF is short for "probability of failure" and is used to compute the likelihood curves for a platform. The program performs the operations defined in equations 4-2 to 4-9. The program uses a nested loop of 5 simultaneous numerical integrations to perform the computations. Table 4-1 shows an example input. Information required includes the C1C2C3 coefficients (discussed below) used to define base shear acting on the platform at various wave heights and currents applied in a particular direction, storm data (the number of hours of storm in that direction and the wave height, current and wave period for each hour), the defining parameters (median and coefficient of variation) for the distributions (base shear estimate, platform capacity, significant wave height and current), information for the numerical integrations, and computation control. The program uses an automated parameter study to determine if the numerical integrations are being performed correctly. Table 4-2 shows an example output for PF. In this case, three separate PF runs were used to estimate the likelihood curve for a range of b's from 0.2 to 2.4. Appendix E provides details on the PF program.

C1C2C3 is used to define the wave force coefficients required for equation 4-3. The program fits a curve (defined by equation 4-3) to the platform base shear as computed by a set of different waves heights and corresponding current. A three-dimensional iteration method is used to determine the best fit coefficients [Press, et al., 1987]. Table 4-3 shows an example input for C1C2C3. Information required includes the platform base shear (computed using the 3-dimensional platform computer model) for each wave height and current pair, and an initial guess or "seed" value for C1, C2 and C3. Table 4-3 also shows the C1C2C3 output which includes resulting C1, C2 and C3 factors as well as an error check. Appendix F provides further details on the C1C2C3 program.

4.4 PROCEDURE FOR DEVELOPMENT OF BIAS FACTOR

The preceding sections have described the overall calibration process, analytical theory and computer codes used by the project. This section describes the step-by-step procedure used to determine the bias factor.

Step 1: Data Collection for a Platform

The data required for platform calibration work includes the platform orientation, water depth, latitude, longitude, structural configuration, and the "observed" condition of the platform following Andrew. As reported by the platform owners, all platforms used in the calibration had no known damage prior to Andrew.

The observed behavior of a platform is identified as:

- Survived (no damage)
- Damaged with locations of damage
- Severely damaged or near collapse
- Collapsed

This observed behavior is used to make a case for the appropriate classification of the platform. The water depth, longitude and latitude are used to determine seastate data from the Andrew hindcast at the actual site of a platform. The orientation of a platform is important to relate the loading directions on the platform.

In addition to this, information of seastates observed at the platform site, the C1, C2, C3 coefficients to determine the base shear for different seastates, the capacity at failure of successive members, and the capacity at formation of the collapse mechanism are needed. Details of the method followed to establish these quantities are described in later steps.

Step 2: Establish Seastate Data

Hindcast data of Andrew in the Gulf of Mexico [Oceanweather, Inc., 1992] is used to establish storm parameters at the site of the platform. The data is available for a large number of grid points across the Gulf of Mexico.

Grid points in the vicinity of platform location are identified. By comparing the differences in water depth, longitude, and latitude at the platform location and nearby grid points, appropriate wave, current, and wind data is obtained. An interpretation scheme provided by Oceanweather (Appendix D) was used to determine the site-specific metocean data when a platform is located between grid points.

The hindcast data important for calibration work includes storm surge, wave direction with storm hour, significant wave height, zero crossing wave period, and current speed along wave direction. In addition, the maximum individual wave height and maximum individual crest height during the storm are required to decide wave criteria for development of a load profile for the static pushover analysis, to establish base shear coefficients, and to determine the likelihood of wave-in-deck loading.

Step 3: Establish Number of Directions for Analysis

Based upon the magnitude and approach direction of significant and maximum wave heights during each storm hour, the orientation of platform, and the observed behavior of the platform (locations of damages/failures, if any), the "zone of importance," or region of maximum wave loading, which would have led to such observations is identified.

The important directions are selected by comparing platform symmetry and platform capacities against wave approach from various directions of the platform, such as broadside, diagonal, or end-on. By comparing the approach angles of seastates (from Step 2) during individual storm hours, and the angle from the north of the key directions for the platform, the number of wave directions important within the "zone of importance" are identified. Additional directions within this zone could be selected to achieve more refined estimates of the required quantities.

Figure 4-3 presents application of this approach on platform ST151K.

Step 4: Determine Coefficients to Define Base Shear

Representative wave and current combinations are run past the platform computer model and the resulting base shears are determined. Additional forces due to wind and the wave in deck are separately computed and vectorially added to the wave and current base shear. The resulting hydrodynamic forces on the platform for various combinations of wind, wave, and current are plotted. Figure 4-4 indicates one such exercise for platform ST151K. Approximately 30 wave runs are required to generate these base shear plots for each wave approach direction.

The C1, C2, C3 coefficients are then determined by using the C1C2C3 program. In some cases, due to significant changes in slope along the curve, several values of C1, C2 and C3 need to be computed for different ranges of wave heights. Repeated runs are sometimes needed to reduce root mean square error between the input values of base shear and those obtained using the generated C1, C2, C3 coefficients.

The resulting coefficients obtained for platform ST151K are shown in Figure 4-4. Several sets of coefficients are established for waves approaching each primary direction of the platform (e.g., broadside, diagonal, end-on).

Step 5: Estimate Base Shear for Different Sea States

Using the seastate data developed in Step 2, analysis directions selected in Step 3, and the base shear coefficients established in Step 4, base shear values are determined for each direction of importance using equation 4-3. This step is required only to estimate the likely change in base shear for different storm hours and to confirm the directions with maximum base shear values.

The resulting base shear generated for each storm hour for the example platform ST151K is shown in Table 4-4.

Step 6: Ultimate Capacity Evaluation

A nonlinear static pushover analysis is performed to determine the load level (base shear) at successive failure of components and at formation of the failure mechanism for each selected direction. The component failures of importance from these analyses are the failure of joints and braces, yielding of legs and piles, and the pullout or plunging of piles. The failure mechanism could form due to multiple failures in the jacket frames, or foundation piles individually or in combination. The governing mechanism for each direction analyzed for a platform is identified.

Step 7: Determine Likelihood Function for an Individual Platform

The probabilities of failure for a platform are then determined for its selected direction. The formulation described in detail in Section 4.3 is followed to establish separately the probability of failure for given sea states during selected storm hours, and the ultimate capacity in each direction. The process is automated by using the PF program.

The uncertainties in distributions of various quantities in equations 4-7 and 4-8 in Section 4.3 are required for evaluating the probability of failure. The following distributions and variances have been assumed for this project:

Item	Distribution	Expected Value	COV
Capacity, R	Log-Normal	per analysis	0.15
Individual Wave height, H/H_s	Forristall	per hindcast	per formula
Error in H_s	Log-Normal	1.0	0.10
Error in current, U	Log-Normal	1.0	0.15
Error in base shear, S	Log-Normal	1.0	0.25 for wave-in-deck case 0.20 for wave-below-deck case

The above values have been used for all platform cases. The resulting base shear distribution is modified to account for breaking wave height. The breaking wave height for shallow water depths has been considered as 0.78 d, where d is the sum of water depth and storm surge [API, 1993]. A maximum of four storm hours has been considered in evaluating the probabilities of failure for each direction.

A detailed sensitivity analysis has been performed by varying the mean and COV of these parameters to evaluate changes in the resulting probabilities of failure. The results of this sensitivity study are given in Appendix C.

The formulation of PF includes a factor, b, which represents a different estimate of structural capacities (resistances) or different ratios of the best estimate of load and capacity per Step 5 and Step 6. The probabilities of failure for each direction of a platform are evaluated for a range of values of b from 0.2 to 2.5.

The values obtained from the PF program for different b values, represent the likelihood function given an observed failure during specific storm hours for each direction for a platform. In addition to this, the distributions of CDF of load, PDF of capacity, and probability of failure for an individual b value can be obtained.

Step 8: Determine Combined Likelihood Function for all Platforms

For "failed" platforms (severe damage leading to salvage or collapse of platform), the most important wave approach direction is established by comparing base shear and capacity estimates for different directions. For the selected direction, the PF program is run to obtain the likelihood function given an observed failure:

$$lk(b \mid failure) = P_f(b) \quad (4\text{-}12)$$

For the "survived" platforms (no observed damage), the likelihood functions from Step 7 for each important direction of a platform are modified as follows:

$$lk(b \mid success\ in\ a\ direction) = 1 - P_f(b) \quad (4\text{-}11)$$

The combined likelihood function for survived platform cases with more than one important direction is then obtained as follows:

$$lk(b \mid success) = \prod_{\substack{direction \\ i}}^{No.\,of\,direction} [lk(b \mid success\ in\ direction\ i)] \tag{4-15}$$

For "damaged" platforms, the likelihood function obtained in Step 7 is modified for a range of load level (α_1b to α_2b), at which the observed damages are predicted to occur per the static pushover analysis. α_1 indicates the ratio of the load level at which the analysis predicts the observed damages to the ultimate capacity of the platform. α_2 indicates the ratio of the load level at which the analysis predicts failure of additional members which were not observed to the ultimate capacity of the platform. The resulting likelihood function for the damage case would then be as follows:

$$lk(b \mid damage) = P_f(\alpha_1 b) - P_f(\alpha_2 b) \tag{4-14}$$

The combined likelihood function of b given observed behavior of n-number of platforms with a combination of x-survivals, y-damages, and z-failures is obtained by direct multiplication of likelihood curves for each individual platform as follows:

$$lk(b \mid new\ observation\ from\ n\ platforms) = \prod_{\substack{platform \\ i}}^{No.\,of\,platforms} [lk(b \mid observations_i)] \tag{4-16}$$

Step 9: Determine Posterior Distribution of B for All Platforms

The combined likelihood function developed for a number of platforms in the last step is then used to establish distribution of bias factor, B, by Bayes theorem of probability theory as follows:

$$f_B''(b) \propto lk(b \mid new\ observation\ from\ n\ platforms) \cdot f_B'(b) \tag{4-17}$$

where $f_B'(b)$ is the prior distribution on B and $f_B''(b)$ is the posterior distribution on B. A normal distribution with a mean of 1.0 and COV of 0.3 for prior estimate of B has been assumed in this project. The 1.0 mean value assumes that existing assessment procedures are "accurate"in terms of predicting platform survival, damage or failure. The 0.3 COV is an initial reasonable estimate for the uncertainty and is similar to that used in other industries. As discussed later, a sensitivity study indicated that the posterior distribution of B is relatively insensitive to the COV of the prior.

The mean and COV of the posterior distribution, $f''_B(b)$ are then determined. The change in the mean value of B from 1.0 (or previous updated value) identifies bias (conservatism or non-conservatism) in the load and resistance recipe.

In this project, the attempt is to establish distribution of a single global bias factor irrespective of the combination of failure types leading to ultimate collapse. Correlations between seastates, load level, and capacities in different directions have been neglected.

4.5 RESULTS OF CALIBRATION ANALYSIS

4.5.1 Case Studies Used in Calibration Work

As previously indicated, thirteen (13) platforms were used for calibration work. Following Andrew, the platforms were inspected by operators for damages. In some cases, the crest levels of waves based on damage and movement of various equipment on the deck were also estimated.

The platforms were classified as follows based upon the observed data supplied by operators:

6 survival (no damage) cases:	ST151K, ST130Q, ST134W, WD90A, MC 311, MC397
3 damage cases:	T23, T25, ST161A
4 failure cases:	ST177B, ST151H, ST130A, T21

Platform ST134W had one damaged brace, but structural analysis did not indicate occurrence of failure of any brace up to the ultimate capacity (the collapse mechanism was the foundation) due to load effects from Andrew alone. Therefore ST134W was considered as a survival case. Platform ST177B was severely damaged and was removed and salvaged. Therefore, it was considered as a failure case.

The selection criteria for these platforms considered variation in physical characteristics to obtain representative cases for a large number of similar platforms. An attempt was also made to select unexpected cases in different categories.

4.5.2 Characteristics of Platforms and Seastate Data

A summary of the important characteristics of the selected platforms is given in Table 4-5. The selected platforms were installed between 1958 and 1991 in water depths from 61 ft to 468 ft.

The Oceanweather hindcast presents seastate data for up to 23 storm hours for each grid-point. Seastate data for each platform site was developed using Andrew hindcast data [Oceanweather, 1992] for selected grid points in the vicinity of the platforms. The selected grid points and interpolation factors used for each platform are given in Appendix D. A summary of the resulting hindcast maximum seastate for each platform is given in Table 4-6. The storm waves in general approached from Northeast, East, Southeast, and South directions. Further description of the platforms is provided below for the three different categories of survival, damage, and failure.

Survival cases: These six platforms were installed between 1963 and 1991 in water depths from 137 ft to 468 ft. The platforms were located in South Timbalier, West Delta, and Mississippi Canyon blocks and were wellhead, quarters, or process type. Three platforms were 4-legged and the remaining three were 8-legged. The leg-pile annulus for three of the platforms was grouted. The three platforms installed before 1964 had K-braces/joints and the remaining three had diagonal or X-braces in the vertical frames. The maximum hindcast wave height for these platforms varied from 50.5 ft to 65.9 ft, with crest heights ranging from 33.5 ft to 41.3 ft. The maximum associated current in the direction of waves varied from 1.33 fps to 3.44 fps. Waves impacted the cellar deck of two of the platforms, but did not impact the main deck. The operator's post storm damage assessment indicated that for some platforms the wave crest was even higher, based on the condition of the stairs, handrails and topside equipment. For some platforms, the operator inferred that the wave height may have been as high as 72 ft.

Damage cases: These three platforms were installed from 1964 to 1969 in water depths from 62 ft to 118 ft. The platforms were located in South Timbalier and Ship Shoal blocks and were hub or production/quarters type. Two platforms were 4-legged, one platform was 8-legged, and each had a grouted leg-pile annulus. The two 4-legged platforms had K-braces/joints in both orthogonal direction frames, whereas the 8-legged platform had K-brace/joints in the end-on frames and diagonal braces in the broadside frames. The maximum hindcast wave height for these platforms varied from 50.3 ft to 58.5 ft, with crest heights ranging from 30.9 ft to 40.2 ft. The maximum associated current in the direction of waves varied from 3.56 fps to 4.25 fps. Waves appear to have impacted the cellar deck of one platform.

Failure cases: These four platforms were installed from 1958 to 1969 in water depths from 61 ft to 142 ft. The platforms were all located in South Timbalier blocks and were hub, wellhead and process type. Three platforms were 8-legged and one was 4-legged, with all having a grouted leg-pile annulus. All 8-legged platforms had K-braces/joints in the end-on frames and diagonal braces in the broadside frames. The 4-legged platform had K-braces/joints in the both orthogonal frames. The maximum hindcast wave height for these platforms varied from 49.7 ft to 60.95 ft, with crest heights ranging from 31.1 ft to 41.8 ft. The maximum associated current in the direction of waves varied from 3.41 fps to 4.26 ft/sec. Waves appear to have impacted the cellar deck of two platforms.

4.5.3 Probability of Failure of Platforms During Andrew:

Base shear coefficients were determined for these platforms following Step-4 described in Section 4.4. Estimates of maximum base shear for all storm hours were made using the base shear coefficients established using the C1C2C3 program. Tables with seastate parameters, base shear coefficients, and expected maximum hindcast base shear during each storm hour are given in Appendix B for all platforms.

The platforms were then analyzed to establish load levels (base shear) at failure of successive members (joints, braces, legs, piles) and at formation of a failure mechanism by following the procedure detailed in Section 3. The results of ultimate capacity analysis for only the most important directions were used in the calibration work. The probabilities of failure for these platforms were then evaluated by using the PF program, and results were obtained for different values of b ranging from 0.2 to 2.5.

A comparison of ultimate capacity and maximum hindcast base shear values for the different directions evaluated and the resulting probabilities of failure at b=1.0 are given in Table 4-7 for all platforms. The ratio of ultimate capacity to expected maximum hindcast base shear varied from 0.78 to as high as 3.69. The probabilities of failure during selected storm hours in a single direction vary from 0.001 to 0.88 for these platforms.

For purposes of classification, a probability of failure of 0.50 has been considered as a target level for defining expected or unexpected failures or survivals. The platform was expected to survive if the probability of failure is less than 0.50. The platform was expected to fail if the probability of failure is more than 0.50. This is obviously a subjective classification and it could be argued that other values of the probability of failure (e.g. 0.20 or 0.40) are more appropriate. The intent for this project was to define a reasonable transition value that can be used to help explain the calibration results. Note that the classification of unexpected survival, expected survival, expected failure, etc. do not impact the final calibration results (i.e., they are for discussion purposes only).

A summary of results along with the expected maximum base shear during Andrew is presented below for the three categories of platforms.

Survival cases: The ultimate capacity results for two directions were used for calibration of behavior of the 5 survived platforms and for only one direction for one platform.

Platform ST151K had maximum waves during storm hours 6 to 8, with waves approaching from the broadside and diagonal directions, with waves from both directions inundating the cellar deck. The first components to fail are a group of K-joints at 2,330 kips in the broadside direction; the ultimate capacity in this direction was 3,500 kips. The hindcast maximum wave force was computed as 4,765 kips. Platform ST130Q had maximum waves during storm hours 6 to 8, with waves approaching from the diagonal direction and waves inundating the deck. The pushover analysis indicated the pile yield as the first component failure at 1,118 kips. Up to the ultimate capacity of 1,265 kips, two piles yield, and pile pullout/ plunging occurs. The hindcast maximum wave force during these storm hours ranges from 964 kips to 1,214 kips. The total probability of failure for these two platforms during Andrew is 0.03 and 0.67. These two platforms are thus classified as **unexpected survivals.**

Platform ST134W had maximum waves during storm hours 6 to 8, with waves approaching from the end-on and diagonal directions. The pushover analysis indicated first yield of one pile section at 1,620 kips and ultimate formation of pile hinges at four piles at 1,923 kips. The hindcast estimate of maximum base shear is 1,307 kips. Platform WD90A had maximum waves during storm hours 7 to 11, with waves approaching from the diagonal and end-on directions. The first failure in the end-on direction occurs at 2,614 kips, at which the K-joints and a large number of braces fail. Pile pullout eventually occurs at the ultimate capacity of 3,267 kips. The hindcast maximum wave force is 2,029 kips. The probability of failure for these two platforms during Andrew is 0.29 and 0.34. Therefore, these two platforms are classified as **expected survivals.**

Platforms MC311 and MC397 platforms are highly redundant platforms with ultimate capacities in the range of 11,566 kips to 20,700 kips. The hindcast maximum base shear for these platforms is estimated to vary between 3,144 kips and 6,382 kips. The ultimate capacity is based upon formation of frame failure or pile failure mechanisms. The probabilities of failure for these platforms are near 0.01. Therefore, these two platforms are classified as **sure survivals.**

Damage cases: The evaluation of wave approach directions along with the platform orientation indicated that only one direction is the most important for the 3-damaged platforms.

Platform T23 in ST52 block had high waves during storm hours 5 to 7, which approached the platform at roughly 115 degrees from true North. The failure of K-joints in the bottom bay on the platform's north and south frames occurred at 901 kips. Thereafter, other K-joints failed and first pile yield occurred at 1,686 kips. The ultimate capacity of the platform in this direction is 2,006 kips (Ru). The hindcast maximum wave force is estimated as 1,092 kips. This platform is located in 63 ft and the breaking wave height computed as 0.78*d (where d includes storm surge) characterized the maximum wave height at the site. The probability of failure for the selected storm hours is 0.01. The observed damages of the platform included failure of the two K-joints reported by analysis at 989 kips load level. The next set of braces failed in the analysis at 1,105 kips. Therefore, the probability of occurrence of damage is evaluated for this platform, when the hindcast load level remained between the pushover load levels 989 kips (0.49 Ru) and 1,105 kips (0.55 Ru). The probability of failure for this condition at b equal to 1.0, is estimated as 0.15. Based upon analysis and observed results, this platform is classified as **expected to survive but likely to have damaged K-joints.**

Platform T25 in SS139 block had high waves during storm hours 6 to 9, which approached in the orthogonal and diagonal directions. The observed failures indicated that damage probably occurred from waves in the orthogonal direction. Therefore, the orthogonal direction was selected for the calibration work. The first failure occurs in K-joints at 671 kips. The ultimate capacity of platform is 1,342 kips (= Ru). The hindcast maximum base shear is estimated as 1,691 kips. The probability of failure is 0.87. Failure of K-joints was observed in the bottom two bays of the North and South frames of the platform. The pushover analysis indicated that these K-joint failures occurred at a base shear of 923 kips. The next set of K-joints in the upper bay failed in the analysis at 1,132 kips.

The probability of failure for the condition in which the pushover load level was between 923 kips (= 0.69 Ru) and 1,132 kips (= 0.84 Ru) is estimated as 0.03 at b equal to 1.0 and as 0.31 at b equal to 1.8. This platform is classified as **unexpected to survive and most likely to have multiple damage of K-joints.**

Platform ST161A had high waves during storm hours 6 and 7, which approached the broadside direction and inundated the cellar deck. The first component failure occurred at 900 kips in the T-joint at Elev. (+)10' and a number of other T-joints at this level failed in compression and tension up to 1,474 kips load level. The first failure of K-joint occurred at 3,670 kips at Elev. (+)10' level and the last K-joint failure occurred at Elev. (+)10' at 4,014 kips. First yield in two deck legs at cellar deck level occurred at 3,583 kips and 3,977 kips. The hindcast maximum wave force is estimated as 3,973 kips. The probability of failure for the selected seastates and the ultimate capacity of 4,426 kips (=Ru) is 0.51. The K-joint failures were observed following Andrew. The Andrew load estimate of 3973 kips is close to the pushover load level at failure of the second K-joint at 4,014 kips (= 0.91 Ru) and lower than the ultimate capacity of 4,426 kips; thus, the observed failures are predicted. The probability of occurrence is determined by using the formulation for damaged platforms given in Section 4.4, and the probability of failure at b equal to 1.0 is 0.07. This platform is classified as **expected to survive and likely to have observed damages.**

Failure cases: The direction which is most likely to have maximum loads during Andrew were analyzed for each of the four failed platforms. The selected directions are given in Table 4-5.

Platform ST177B had maximum waves during storm hour 6 at 88 degree from true North, with waves approaching the diagonal direction at 22.5 degree from broadside. The pushover analysis indicated failure of a number of K-joints in the end-on frames at 2,452 kips at bottom bay joints, and 2,942 kips at Elev. (-)28' joints. The elev. (-)65' joints failed at 3,677 kips. First pile yield occurs at 3,677 kips and the ultimate capacity of platform due to frame failure and pile hinge formation occurred at 4,168 kips. The observed damages indicate failure of K-joints at Elev. (-)28' and at Elev. (-)65'. The hindcast maximum base shear with inundation of deck is 5,150 kips. The probability of failure for the selected storm hours is 0.77. This platform is classified as an **expected failure.**

Platform ST151H had maximum waves during storm hours 6 to 8, which approached diagonal direction. The pushover analysis indicated first failure of K-joint in the end-on frames at 2,666 kips and successive failure of several diagonals in the broadside frames leading to ultimate failure of the platform at 3,999 kips. The first yielding of a pile occurred at 3,999 kips. The hindcast maximum base shear for this direction is estimated as 4,206 kips with inundation of the cellar deck. The probability of failure during the selected storm hours is 0.69. This platform is classified as an **expected failure.**

Platform ST130A had maximum waves during storm hours 6 to 8, which approached from the diagonal direction. The pushover analysis indicated that first K-joint failure occurred at 1,300 kips and first yield of pile occurred at 1,800 kips, with the ultimate capacity of platform at 3,000 kips. The maximum hindcast base shear is estimated as 2,779 kips. The probability of failure for selected storm hours is 0.53. This platform is classified as a **likely failure.**

Platform T21 in ST72 block had maximum waves during storm hours 5 and 6 which approached from the orthogonal direction. The pushover analysis indicated failure of K-joints starting at 1,050 kips, first yielding of pile section at 1,633 kips, with the platform capacity due to frame failure at 1,984 kips. The maximum hindcast base shear is computed as 1,615 kips. Although the probability of failure for the selected storm hours is 0.40 (less than 0.50), it was decided to categorize this platform as a likely failure due to additional unknowns for this platform such as potential pre-existing damage, site-specific soil conditions, and shallow water wave forces. This platform is classified as a **likely failure.**

4.5.4 Likelihood Function Development

Table 4-6 presented the most important directions plus the parameters for the storm hour with the hindcast maximum seastate in that direction. The parameters for other storm hours and base shear coefficients are given in the tables for each individual platform in Appendix B. Table 4-7 presented the ultimate capacity for the directions selected.

As previously discussed, the probability of failure analysis was performed by using the PF program. The platforms were analyzed for the directions identified in Table 4-7 for seastates during a maximum of four storm hours. The distributions summarized in Step 7 of Section 4.4 were used for all the platforms.

The analysis was performed for a number of values of b from 0.2 to 2.5. The integration limits and number of integration points in the distributions of various parameters to be used in the program would vary with change in the value of b. The program determines the optimized integration limits and points for various parameters for given values of b. Such optimized integration limits were established for a number of b values and the program was run using these to determine probability of failure (P_f) for a large number of b values. A plot of resulting values of P_f for different b values is known as the likelihood function given an observed failure for a specific direction of a platform.

The likelihood function for the platform was then obtained by modification of individual likelihood functions given an observed failure for a direction, to account for the effect of multiple loading directions, observed survival, and observed damage cases. The details of arriving at different likelihood functions were given in Step-8 of Section 4.4.

Figure 4-5 presents the individual likelihood functions for the six success (survival) cases. Note that the likelihood functions for five platforms represent the combined effect of two directions selected for each case. The combined likelihood function in the event of observing success of all six platforms (6 out of 6 success event) is given by the bold line in this figure. The positioning of the likelihood functions for the six platforms indicate distinct differences and justifies the classification of platforms reported in Table 4-7.

Figure 4-6 presents the likelihood functions for the three damaged platforms. The likelihood functions from PF program are modified for the probability of hindcast load level to lie between 0.49 Ru to 0.55 Ru for T23, between 0.69 Ru to 0.84 Ru for T25, and between 0.91 Ru and 1.0 Ru for ST161. The damage likelihood function takes the form of a distribution function compared to the cumulative distribution function form for the survival and failure cases. The combined likelihood function, given damages in three platforms (three out of three damage event), is obtained by multiplication of the individual functions.

Figure 4-7 presents the individual likelihood functions for the four failure cases. Only one direction was analyzed for these platforms. The cumulative likelihood function given 4 failures (four out of four failure event) has been determined by multiplication of the four individual distributions.

Figure 4-8 represents the change in likelihood function with observing successive failure cases. This figure shows the importance of including all 4 observed failure cases and that further shift in the combined likelihood function for failure cases only may not be significant by additional new failure cases. The likelihood function in bold indicates the event of observing 4 out of 4 failures. The 4 failure cases limit the upper end of the function at b equal to 1.5.

Figure 4-9 demonstrates the effect of including observed damage cases. The inclusion of ST161A limits the lower end of the combined likelihood function for failure cases at b equal to 0.3 and its peak lies at b of 0.7. By addition of the other 2 damage cases, the peak value and lower limit of combined function shift to right. The peak of the combined likelihood function when 4 failure cases and 3 damage cases are observed becomes 1.04 and it ranges between b of 0.7 and 1.5.

Figure 4-10 demonstrates the effect of observing successive survival cases. The inclusion of platforms MC397 and MC311 (sure survivals) does not shift the combined likelihood function for the seven observed failure and damage cases during Andrew. By including WD90A and ST134W (expected survivals), the shift in peak value is marginal. The peak value and combined function shifts to right primarily due to ST130Q and ST151K cases (unexpected successes). The peak of combined likelihood function for the observed behavior of all 13 cases is at b of 1.17 and it ranges between b of 0.9 to 1.7.

Figure 4-11 demonstrates the effect of successively including each group of failure, damage, and survival cases. Note that the observed failure cases and damage cases during Andrew limit the upper end and lower end of the combined likelihood function with a peak near b of 1.04 and ranging between b of 0.7 and 1.5. The effect of including the unexpected observed success cases, ST130Q and ST151K, shifts the peak from 1.04 to 1.17 and the ultimate combined likelihood function range between b of 0.9 and 1.7.

4.5.5 Development of Posterior Distribution of Bias Factor

The posterior distribution of bias factor is established by following the procedure given in Step 9 of Section 4.4. A prior distribution of bias factor, B is assumed as a Normal distribution with mean of 1.0 and COV of 0.3. The cumulative likelihood functions developed in Figures 4-5 to 4-7 for survival, failure, damage, and in Figure 4-11 for all combined cases are then multiplied with the prior distribution of B to obtain the respective posterior distributions of B.

Figure 4-12 presents the prior distribution of B, the change in posterior distribution with inclusion of each observed success case, and the posterior distribution when all six success cases are included. It is noted that platforms MC397 and MC311 have negligible effect, platforms ST134W and WD90A have moderate effect, and platforms ST130Q and ST151K have a significant effect in shifting the posterior distribution. The mean and COV of posterior distribution of B given the six success cases are 1.44 and 0.14, respectively. The posterior distribution given only six successes indicates that the assessment process is conservative.

Figure 4-13 presents the prior distribution of B, the change in posterior distribution with inclusion of each damage platform case, and the ultimate posterior distribution when all three damage cases are included. The mean and COV of posterior distribution of B given the 3 damage cases are 1.26 and 0.13, respectively.

Figure 4-14 presents the prior distribution of B, the change in posterior distribution with inclusion of each observed failure case, and the ultimate posterior distribution when all four failure cases were observed. It is noted that all four platforms have influence in shifting the posterior distribution of B. The mean and COV of posterior distribution for the given four failure cases are 0.69 and 0.27, respectively. The posterior distribution given only four failures indicates that the assessment process is unconservative.

Figure 4-15 presents the prior distribution of B, posterior distributions generated in Figures 4-12 to Figure 4-14 for survival, failure, and damage cases respectively, and the ultimate posterior distribution given observed information for all 13 platforms. The mean and COV of the ultimate posterior distribution of B given information for 13 platforms are 1.19 and 0.10, respectively.

4.5.6 Sensitivity of Change in Parameters/ Assumptions on the Posterior Distribution

During the course of this project a number of sensitivity studies were performed to understand the influence of the various parameters on estimates of the probability of failure. A summary of the sensitivity studies performed to evaluate the effect on the posterior distribution of B is presented in this section. Results of other sensitivity studies are provided in Appendix C.

- **Case 1: Variation in COV of prior distribution of B**

  Figure 4-16 shows the effect of variation in COV of the prior distribution of B from 0.3 to 0.2 and 0.4. The resulting posterior distributions show marginal change. The mean B changes to 1.16 and 1.20 for COV of prior of B equals 0.2 and 0.4 respectively. The posterior COV of B reduces to 0.09 for prior COV of 0.2. Therefore, the assumed 0.30 COV of the prior distribution of B is reasonable.

- **Case 2: Reduction in one major case from each category**

  Figure 4-17 presents the effect on posterior distribution of B when a single platform is removed from any of the three categories.

  The results indicate that removal of either ST151K (a survival case) or T25SS139 (a damage case) has nearly the same effect on the posterior distribution of B. The mean of B reduces to 1.11 for either of the twelve-platform groups from 1.19 for the base case group (13 platforms). Note that the mean of the prior of B shifts from 1.0 to 1.36, when only ST151K is included (Figure ES-2), whereas the shift in mean is from 1.11 to 1.19 when the effect of the 12 other platforms has been included. With the removal of T21 ST72, a failure case, the mean of B increases from 1.19 to 1.27. The change in mean of B is roughly 7% for either of the three cases.

  The COV of B increases marginally from 0.10 to 0.11.

- **Case 3: Removal of three damage cases**

  Figure 4-18 shows the effect of removal of all three damaged platforms on the posterior of B. The mean of B reduces from 1.19 (13 platforms) to 1.11 (10 platforms), i.e, approximately 6 percent reduction.

  The COV of B increases to 0.12.

- **Case 4: Assuming 13 more similar observations are available**

  Figure 4-19 shows the effect of inclusion of additional platforms. It is assumed for this demonstration that there are an additional 6 success, 4 failures, and 3 damage cases for platforms similar to those made available for this project. The posterior of B developed in Section 4.5.5 becomes a new prior with a mean of 1.19 and a COV of 0.10. The posterior distribution for a total of 13 similar additional platforms changes marginally with mean and COV of 1.20 and 0.07 respectively. Note that the distribution of B becomes more peaked with the addition of more platforms.

This figure also demonstrates the effect of including an additional 6-survival cases or 3-damage cases or 4-failure cases. The shift in mean of B for such cases is between 3 to 10 per cent from 1.19 and COV of B reduces marginally to 0.08 – 0.09. In case of including mixed group of additional platforms, the shift in posterior of B will be marginal.

4.6 APPLICATION OF CALIBRATION RESULTS TO OTHER PLATFORMS

The resulting posterior distribution of "B" in Section 4.5, which represents bias in the assessment of the ratio of ultimate capacity to maximum environmental load (Ru/ S), can be used to update the safety index or probability of failure for a platform. The distribution of B presents the effect of introducing modeling errors or Type-II uncertainties, in the various assumptions and simplifications made in the load and strength modeling recipe and analysis techniques followed.

The following describes the steps needed to evaluate the updated annual probability of failure, when the effect of modeling errors established in this project is included. This process is applied to an example platform, ST151K, with an assumed annual maximum seastate distribution.

Step 1: Establish Annual Maximum SeaState Data

The distribution of annual maximum significant wave height, Hs, and associated current and wind magnitudes are needed for different directions of importance to the platform.

For the example application, a log-normal distribution for annual maximum Hs with a median of 17.5 ft., and COV of 0.314, for a location in the Gulf of Mexico in water depth of 137 ft. and storm tide of 4 ft. has been assumed. This distribution has been provided by Dr. Chuck Petrauskas (Chevron) and is only intended for illustration of the updating process. It is not intended for use elsewhere. This distribution applies to the omni-directional wave height. It is assumed that the storm duration is 3 hours which is equivalent to approximately 800 waves. The above distribution for Hs was used for hour 2 of the storm and lower values were used for median of Hs for hour 1 and hour 3 of the storm (based on 7 percent typical reduction in Hs noted in Andrew Hindcast data). The associated current has been assumed to have a median value of 2.1 knots with a COV of 0.15.

The above annual maximum seastate was used for location in 137 ft. water depth in the South Timbalier field, and platform specific seastate data was developed considering orientation of the 8-legged platform ST151K for three approach directions: broadside, diagonal, and end-on. The guidelines given in Section 2.3.4 of API RP 2A were used to develop seastate data for the different directions.

On this basis, the median Hs for storm hour 2 was determined as 16.1 ft, 17.5 ft, and 16.28 ft for broadside, diagonal and end-on directions, respectively. The median value of the current was determined as 1.97 kt, 0.88 kt, and 0.71 kt for broadside, diagonal and end-on directions, respectively.

Step 2: Determine Coefficients to Define Base Shear

The C1, C2, C3 coefficients described in Section 4.4 are required for selected directions of the platform.

For the example platform (ST151K), the base shear coefficients developed for estimation of the posterior distribution of "B" were used.

Step 3: Determine Ultimate Capacity of Platform

The ultimate capacity of the platform using the pushover load reflecting the annual maximum seastate established in Step 1 is required for the selected directions. The details of the various assumptions made in modeling of platform and of the pushover analysis process are given in Chapter 3.

The ultimate capacities determined for the example platform in Section 3.4 as 3,500 kips for the broadside and diagonal directions and as 3,900 kips for the end-on direction were used.

Step 4: Determine Failure Probability versus b

The conditional probability of failure given b, $P[S > Rb \mid b]$, for a range of b values for each selected direction of the platform is required (see equation 4-18 below). The details of development of this probability were given in Section 4.3 and 4.4.

The failure probability functions were obtained for the example platform for the three directions using the PF program. The median values of Hs and current developed in Step 1 were used. A log-normal distribution was assumed for the "error" in Hs with median of 1.0 and COV of 0.314 (based upon the previously noted Gulf of Mexico data). The Forristal distribution was used for the individual wave heights. In addition, the distributions of capacity, error in base shear, and error in current as given in Section 4.4 were used. The resulting variation in failure probability versus b is shown in Figure 4-20.

Step 5: Determine Probabilities of Failure

Probabilities of failure for three cases were used to illustrate the variation in its magnitude due to the updating process. The three cases are identified as follows:

- The probability of failure, P_f, without including modeling uncertainties
- The probability of failure, P_f' with the prior distribution of B
- The posterior probability of failure, P_f'' with the posterior distribution of B

The first value reflects the probability of failure following the conventional procedure (e.g., like that used in the LRFD RP2A development); the second value reflects the explicit inclusion of uncertainty associated with modeling assumptions; and the third value reflects the updated probability of failure after the incorporation of the information about modeling contained in the Andrew experience. The formulations for the three quantities for a specific direction of the platform are given as follows:

$$P_f = P[S > R] \tag{4-18}$$

$$P_f' = \int P[S > Rb \mid b]\, f_B'(b)\, db \tag{4-19}$$

$$P_f'' = \int P[S > Rb \mid b]\, f_B''(b)\, db \tag{4-20}$$

where

$P[S > R]$ represents the probability of failure for given S and R distributions and following the procedure illustrated in Section 4.3, but using the distributions given here.

$P[S > Rb \mid b]$ represents the conditional probability of failure given b, i.e., the variation of probabilities of failure for different fixed values of b.

$f_B^{\prime}(b)$ represents the prior distribution of B assumed in the Andrew-JIP (mean 1.0, COV 0.30).

$f_B^{\prime\prime}(b)$ represents the posterior distribution of B established in the Andrew-JIP, reflecting the experience during Andrew and the model calculated loads and capacities using a defined recipe.

The total probabilities of failure for a platform can be conservatively approximated by:

$$\text{Total } P_f = 1 - \prod [1 - P_f] = \sum P_f \qquad (4\text{-}21)$$

$$\text{Total } P_f^{\prime} = 1 - \prod [1 - P_f^{\prime}] = \sum P_f^{\prime} \qquad (4\text{-}22)$$

$$\text{Total } P_f^{\prime\prime} = 1 - \prod [1 - P_f^{\prime\prime}] = \sum P_f^{\prime\prime} \qquad (4\text{-}23)$$

where the product or sum is over the directions. A lower bound estimate is the maximum P_f in any direction.

The above formulations were applied to the example platform. The plots of the resulting conditional probability of failure, prior distribution of B, and posterior distribution of B are given in Figure 4-21 for the broadside direction. The following annual probabilities of failure were obtained for the three directions:

	Broadside	Diagonal	End-on
P_f	0.037	0.025	0.007
$P_f^{\prime}$	0.071	0.044	0.018
$P_f^{\prime\prime}$	0.022	0.016	0.004

The total annual probability of failure for the platform for the three cases is thus obtained as follows:

P_f = **0.07** (without modeling uncertainties)

P_f' = **0.13** (with prior B)

P_f'' = **0.04** (with posterior B)

From the above results obtained for the example platform, the following are observed:

- The probability of failure (P_f') with prior distribution of B, i.e., including (unbiased) modelling uncertainties, will always be higher than the "simple" prior probability of failure (P_f)

- Given the posterior distribution of B established in the Andrew JIP, with median greater than 1 and a reduced COV, the posterior probability of failure P_f'' will always be lower than the prior probability of failure (P_f').

- In this case, the posterior value, P_f'' is also lower than the simple value, P_f. The primary cause is the median greater than 1; the posterior COV of B is small so it has little effect compared to the random variabilities, e.g., in the annual H_s value or in the C_D coefficient.

4.7 SUMMARY OF CALIBRATION WORK

The calibration work aimed at development of a basis to improve estimates of probability of failure or safety index of a platform. The updating is made possible by establishing a bias factor (B) and its distribution, based upon data from the observed behavior of 13 platforms during Andrew. The probability of failure is then obtained for each platform and Bayes formula of probability theory is used to establish the posterior distribution of B.

Details of the theory developed for calibration of behavior with best estimate from analysis were provided in Section 4.3. Note that in the evaluation of the probability of failure, the data for only a few hours with significant storm was found to be important in this formulation. Appropriate distributions for various parameters were assumed and their effect on estimates of probability of failure were evaluated by sensitivity studies.

The procedure followed for development of bias factor (B) and its distribution is given in detail in Section 4.4. The method presented signifies importance of obtaining platform information as accurate as possible (i.e., "crisp" data) for use in establishing the bias factor.

The ultimate capacity evaluation followed the recipe presented in Chapter 3 for load and capacity estimates, and for various assumptions in modeling and analysis. The load levels at failure of successive members and formation of collapse mechanism indicate that more refined models are needed for some components.

The procedure was applied to 13 selected platforms installed during 1958 to 1991 in water depths ranging from 61 ft to 468 ft. Out of these 13 platforms, 6 survived, 3 were damaged, and 4 failed. The configurations of the platforms varied from 4 to 8 legs, with K-joint, diagonal, and X-brace framing. Waves appear to have inundated cellar decks of 5 of the platforms based on the hindcast maximum wave heights.

Both unexpected survival cases had K-joints. The formula used to evaluate the capacity of the K-joints gives a lower bound result. In addition, both platforms had waves in the deck. These deck wave forces evaluated based on API preliminary deck force guidelines (Table 3-1) are relatively conservative. These could be reasons why the platforms were "unexpected" successes.

The two expected survival cases had a hindcast maximum base shear higher than the load level of failure of the first component. The first component failure occurs in platform ST134W due to first yield in pile section, and for platform WD90A due to failure of one K-joint. These represent lower bound capacities. These could be the reasons for not observing any damages in these platforms.

The two sure survival cases followed new design practices with highly redundant X-braces in the vertical frames. These platforms were designed to newer standards for working stress design. Therefore, these platforms had very low probability of failure and were expected to have survived, undamaged.

The expected failure cases had lower capacity diagonals and K-braces and a low ratio of ultimate capacity to expected Andrew load. Waves inundated the cellar decks of two platforms. These high loads probably led to the observed platform failures.

The two likely failure cases had low margin between expected Andrew load to ultimate capacity, and the analysis indicated failure of a large number of K-joints. The first K-joints failed at a fairly low load. The low margin, combined with multiple failure at the K-joints, probably led to the observed platform failures.

The two damaged platforms were determined analytically to survive. The results indicate high margins for these two platforms with lesser damage. One platform had an expected Andrew load higher than its ultimate capacity and it had significant observed damage. All three platforms had K-joint failures. Lower bound capacity formulas were used for K-joints, which could be the reason for the lower capacity estimate.

The likelihood functions and the posterior distributions indicate that the observed failures during Andrew have nearly equal importance in shifting the distributions. The observed damage case of T25 has more effect in shifting the distributions than other two damage cases. The two unexpected success cases included (ST151K, ST130Q) have more effect in shifting the mean value of B than the other success cases.

The sensitivity analysis results indicated that assumed prior of B is acceptable, as reasonable variation in COV from 0.3 would shift the posterior mean by only 2 per cent. The sensitivity analysis indicated that the mean of B would change by roughly 7 per cent when a single platform is removed from any of the three cases (survived, damaged, failed), and when a mixed group of new platforms are considered the shift in mean of B would be marginal.

Based on the results presented in this section, it may be important to give different weighting to the various platforms used, so that the resulting posterior distribution is independent of the number of platforms. A very large number of platforms may seem to be ideal to include in such a study, but it would be difficult from practical standpoint.

A favorable bias factor, B, with a mean of 1.19 and COV of 0.10 has been established by comparison of analytically predicted behavior with the observed behavior in Andrew. This bias factor reflects conservatism in load and resistance recipe, and pushover analysis techniques followed in this project.

Application of this bias factor to an example platform resulted in a reduced annual probability of failure of the platform.

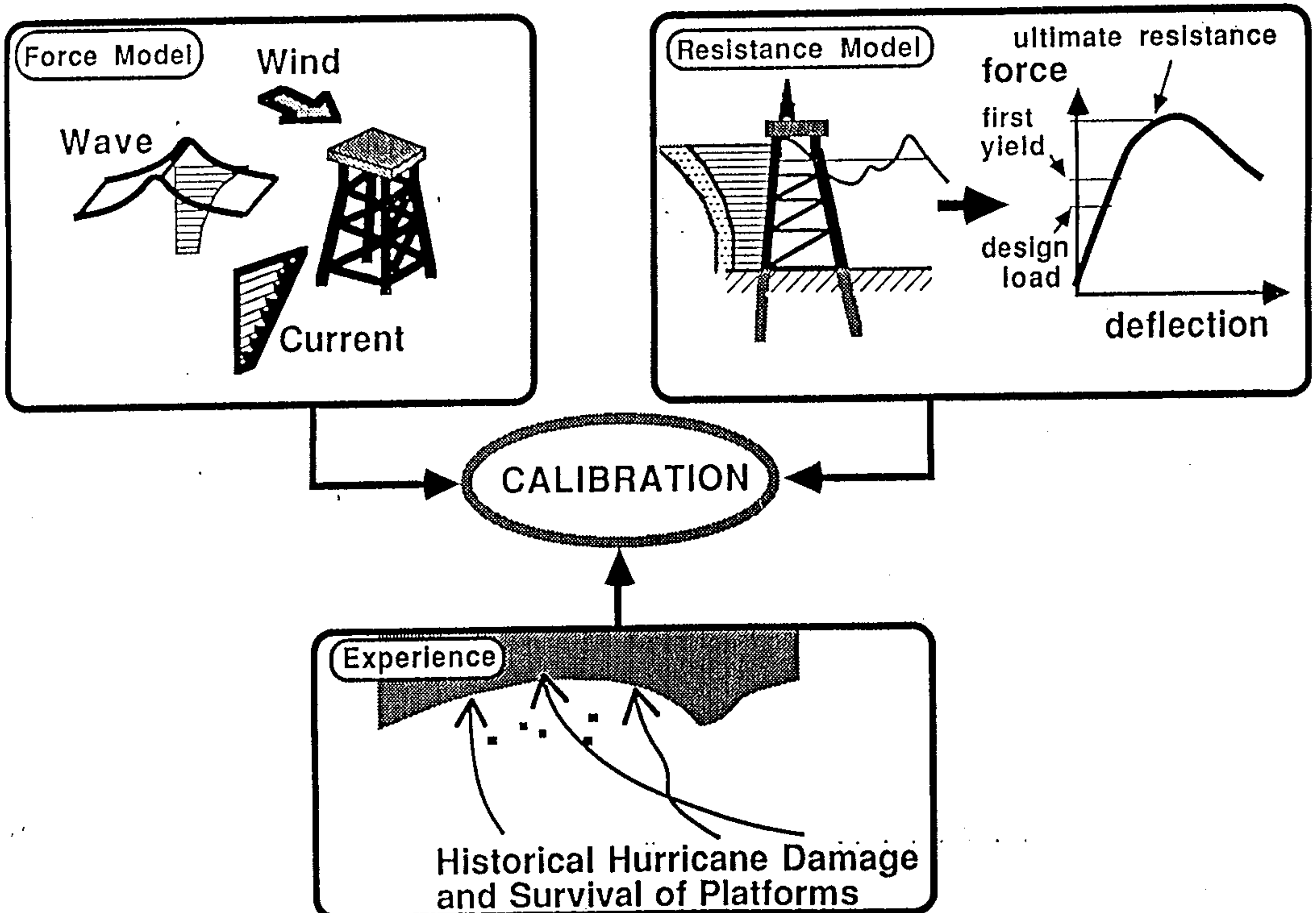


Figure 4-1 Components Required for Calibration (Petrauskas, 1992)

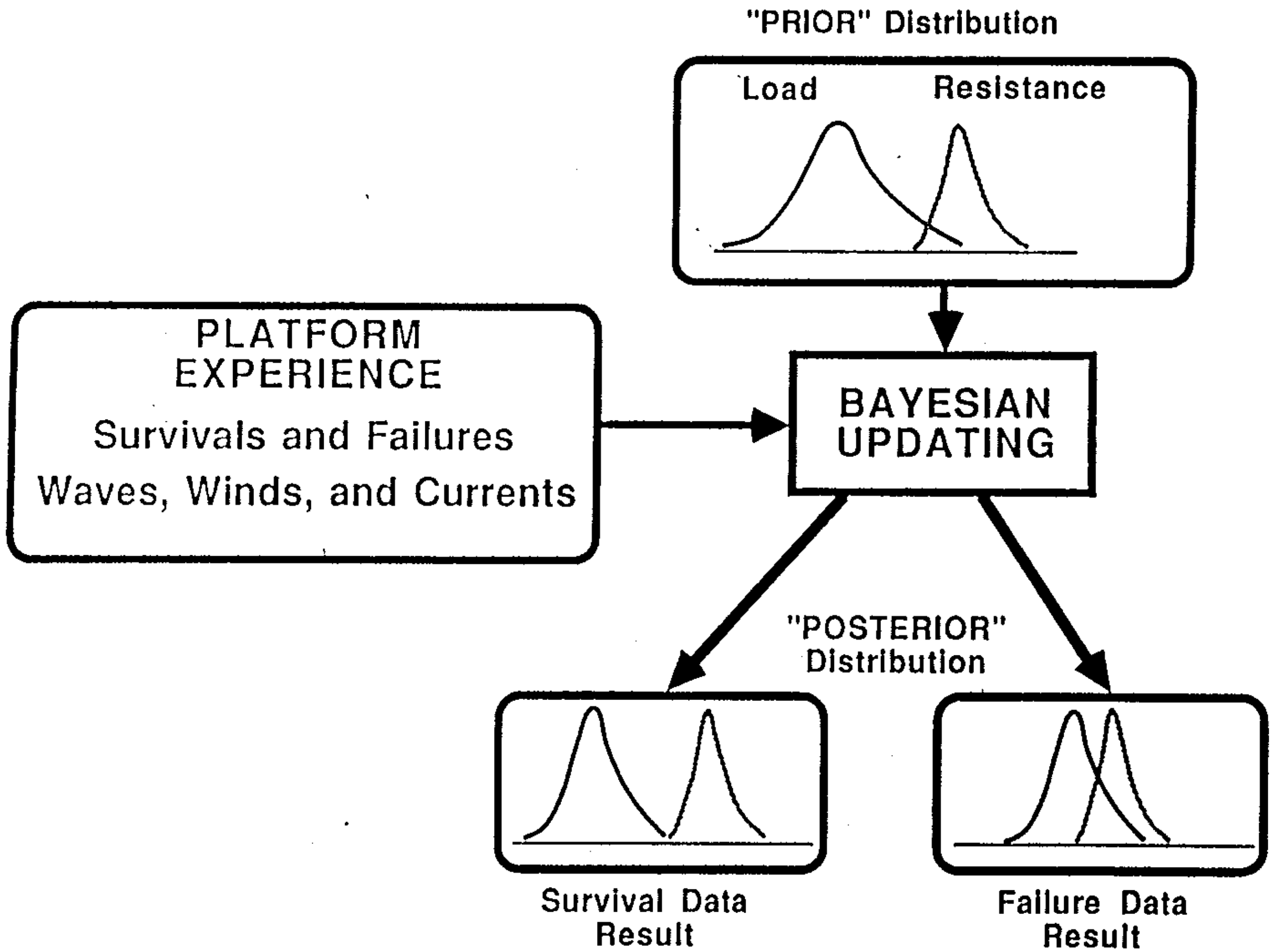


Figure 4-2 Bayesian Updaing - General Description (Petrauskas, 1992)

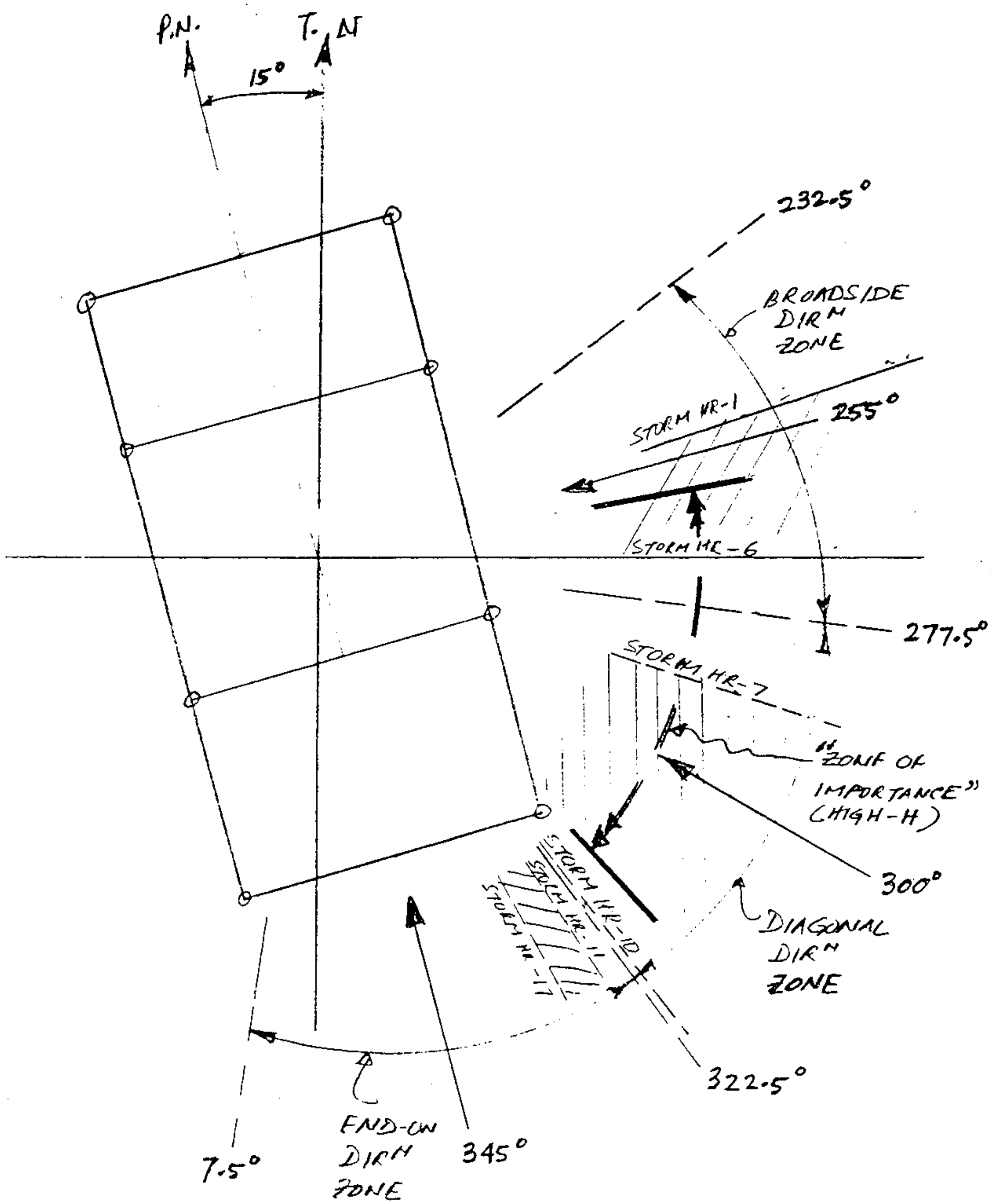
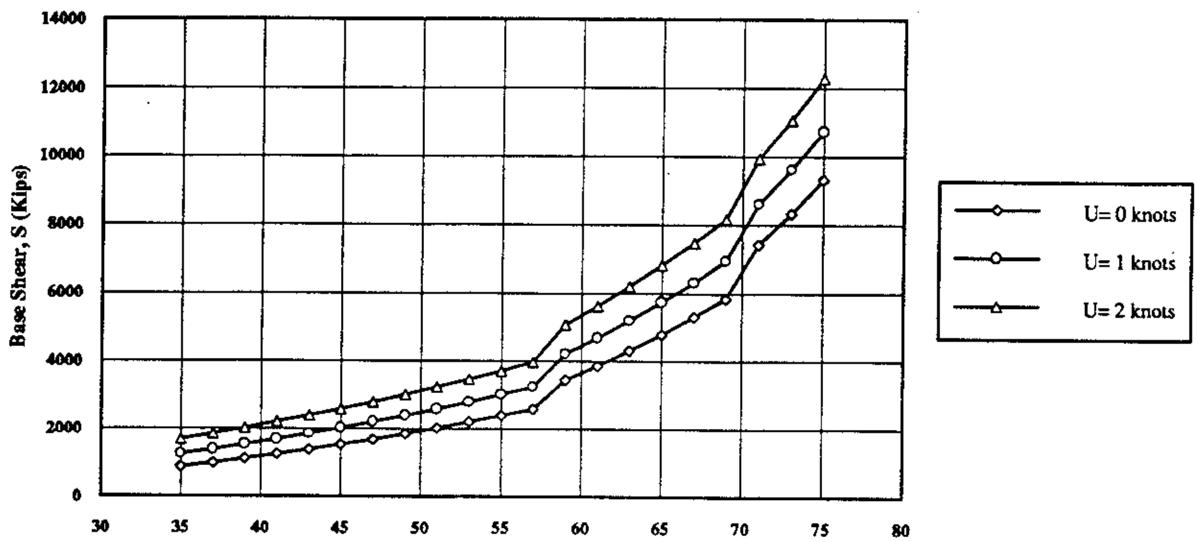


Figure 4-3 Storm Approach Directions and Analyzed Directions - ST151K

Wave Height vs Base Shear: ST151K, Broadside Direction



C1, C2, C3 for Platform ST151K

Direction	Wave Height Range ft.	C1	C2	C3
Broadside	35 to 57	0.301	5.99	2.24
	58 to 69	3.56E-03	3.60	3.38
	70 to 75	1.54E-04	2.59	4.15
Diagonal	35 to 57	0.169	6.21	2.36
	58 to 69	2.87E-03	4.10	3.38
	70 to 75	2.28E-02	4.57	2.90
End-On	35 to 57	0.154	5.33	2.36
	58 to 69	6.23E-03	3.86	3.17
	70 to 75	1.80E-02	4.05	2.92

Figure 4-4 Base Shear Coefficients - ST151K

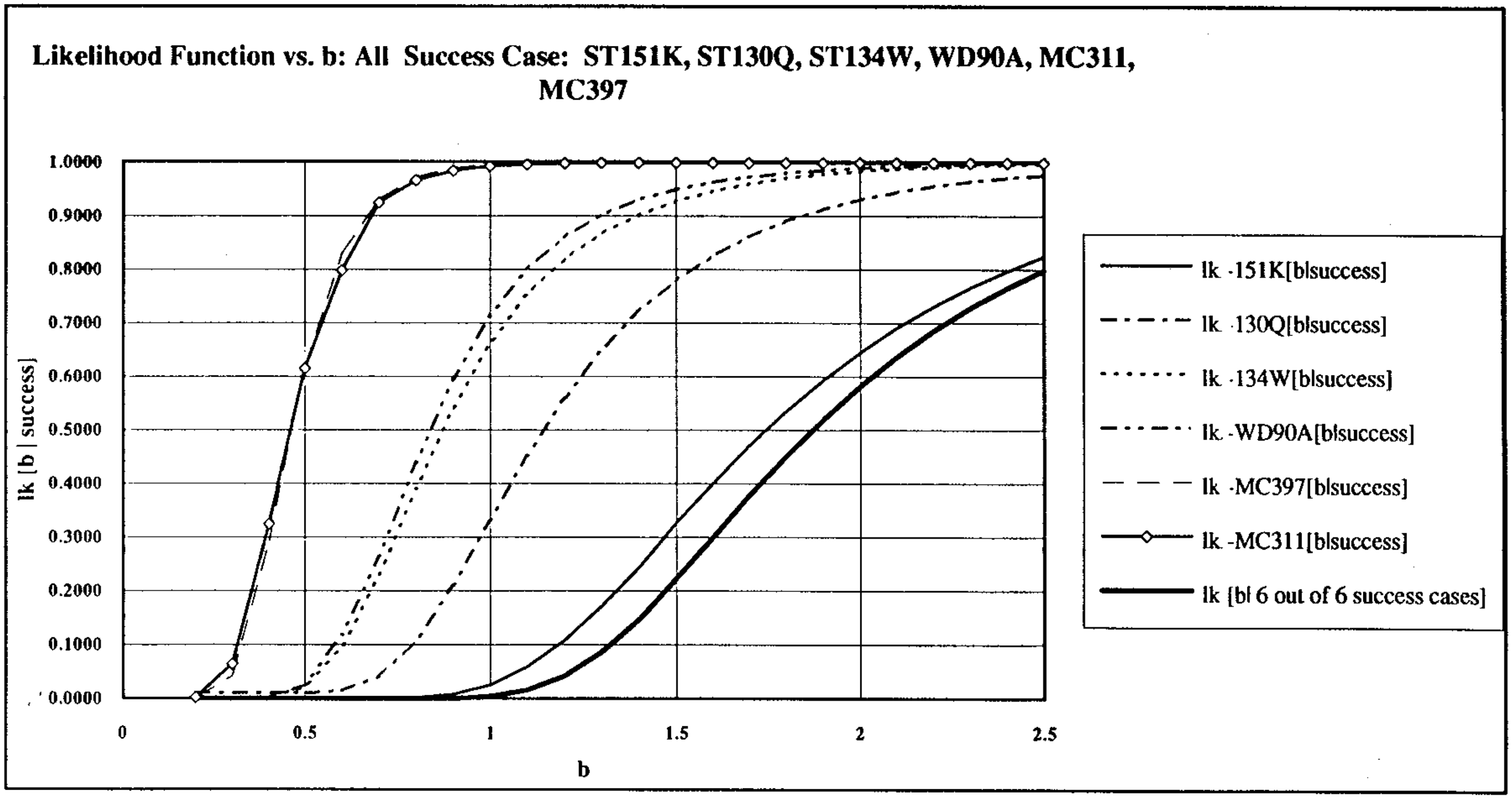


Figure 4-5 Likelihood Functions - Success Cases

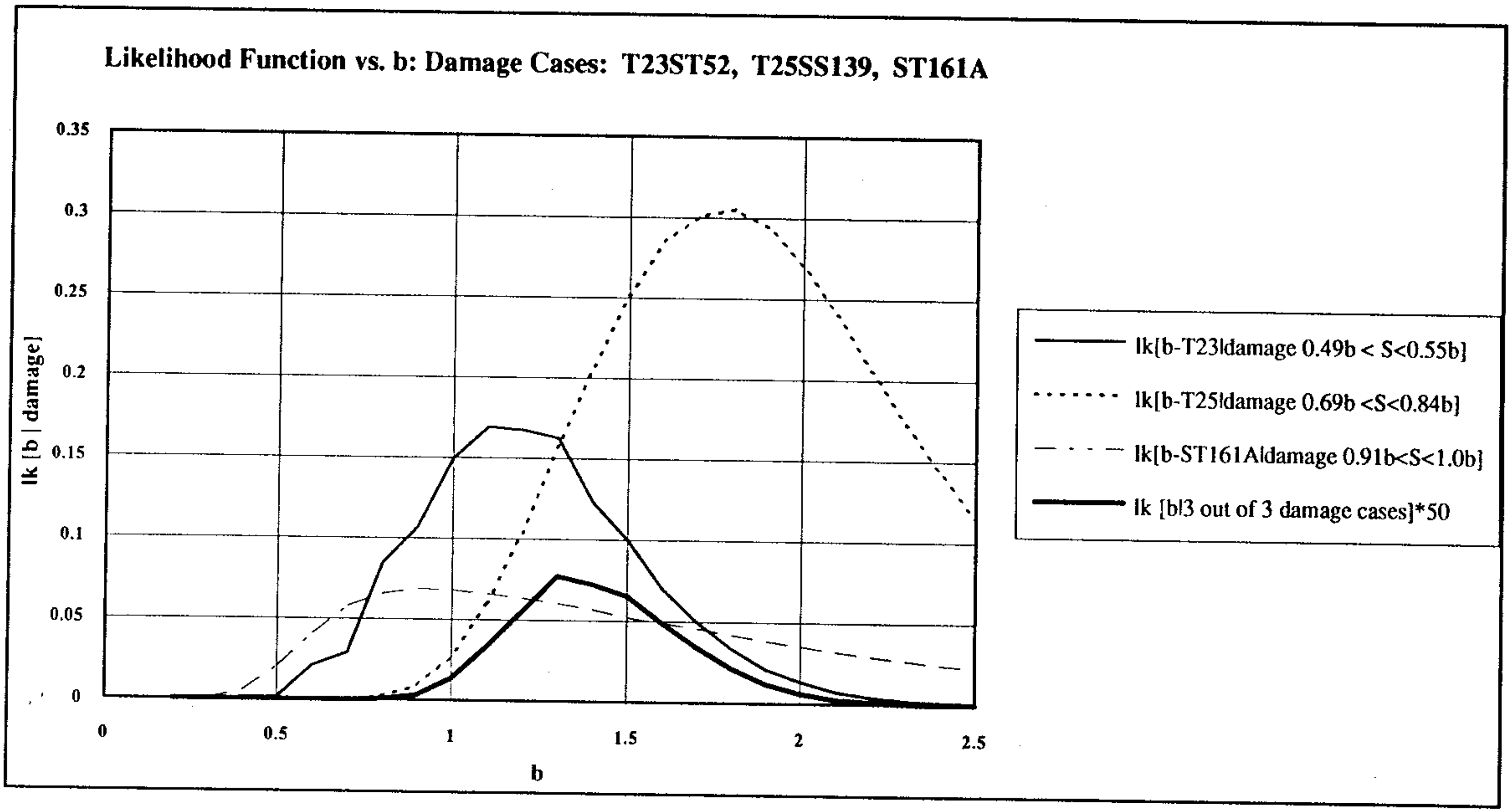


Figure 4-6 Likelihood Functions - Damage Cases

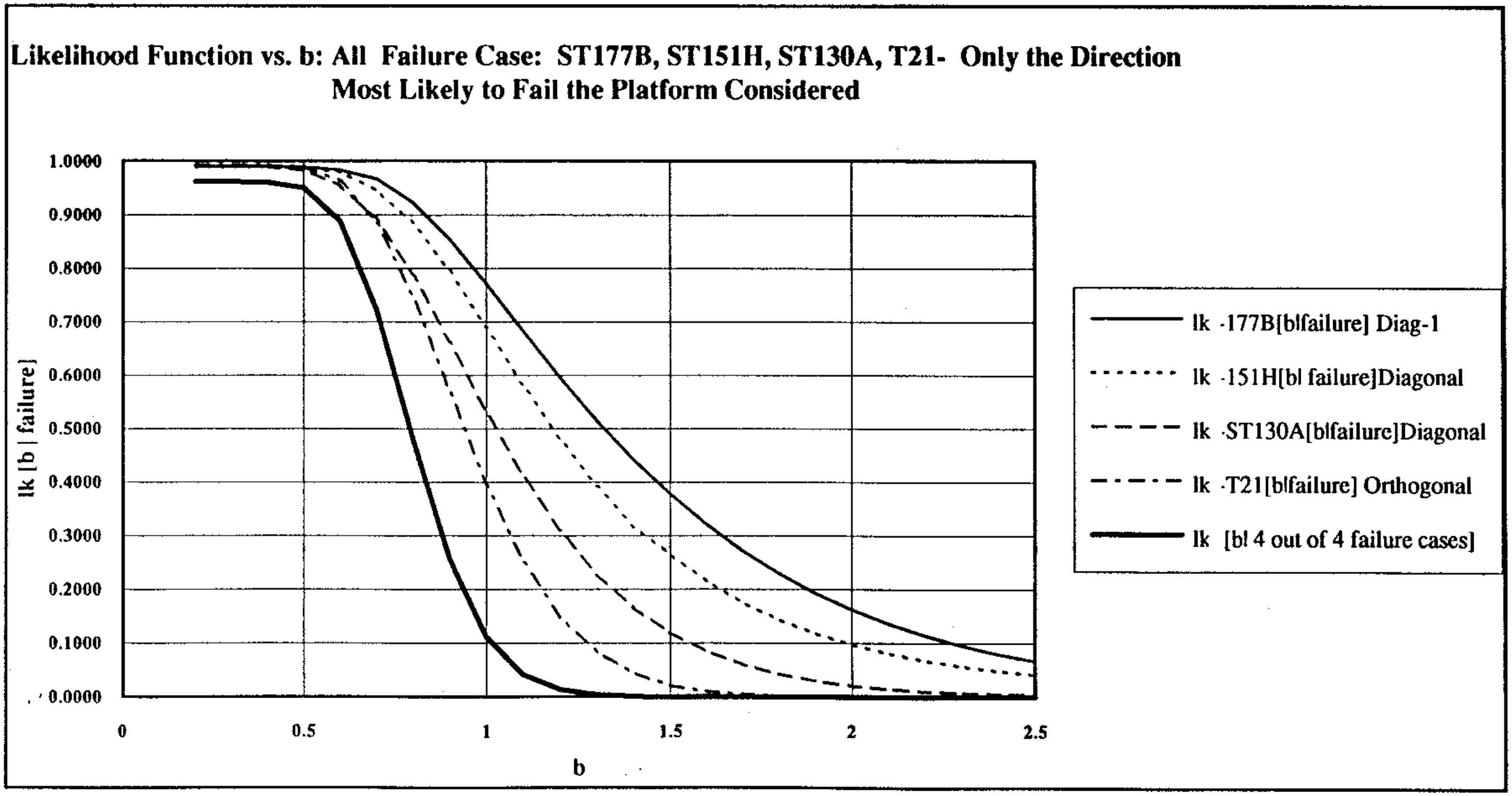


Figure 4-7 Likelihood Functions - Failure Cases

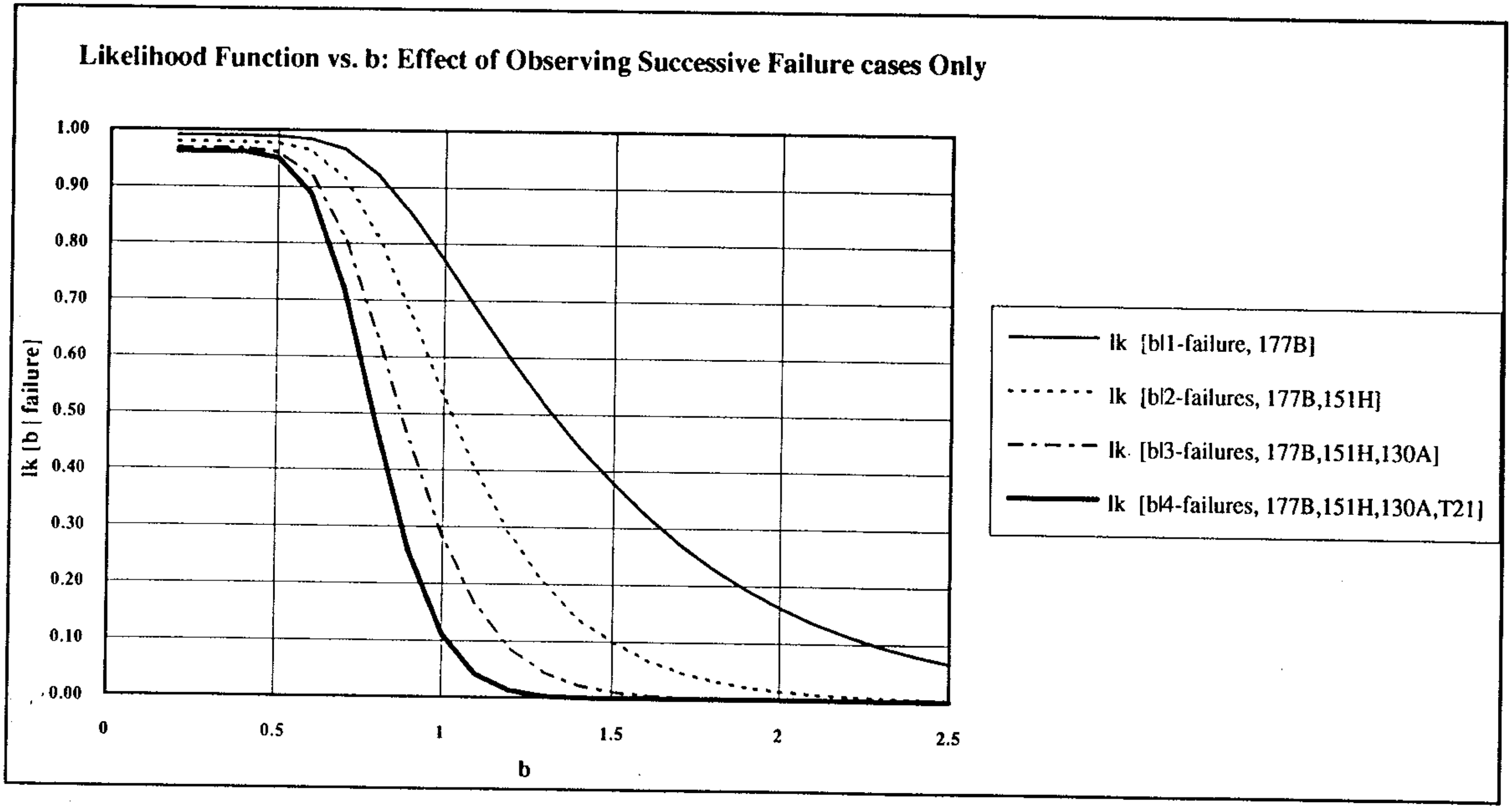


Figure 4-8 Combined Likelihood Functions - Successive Failure Cases

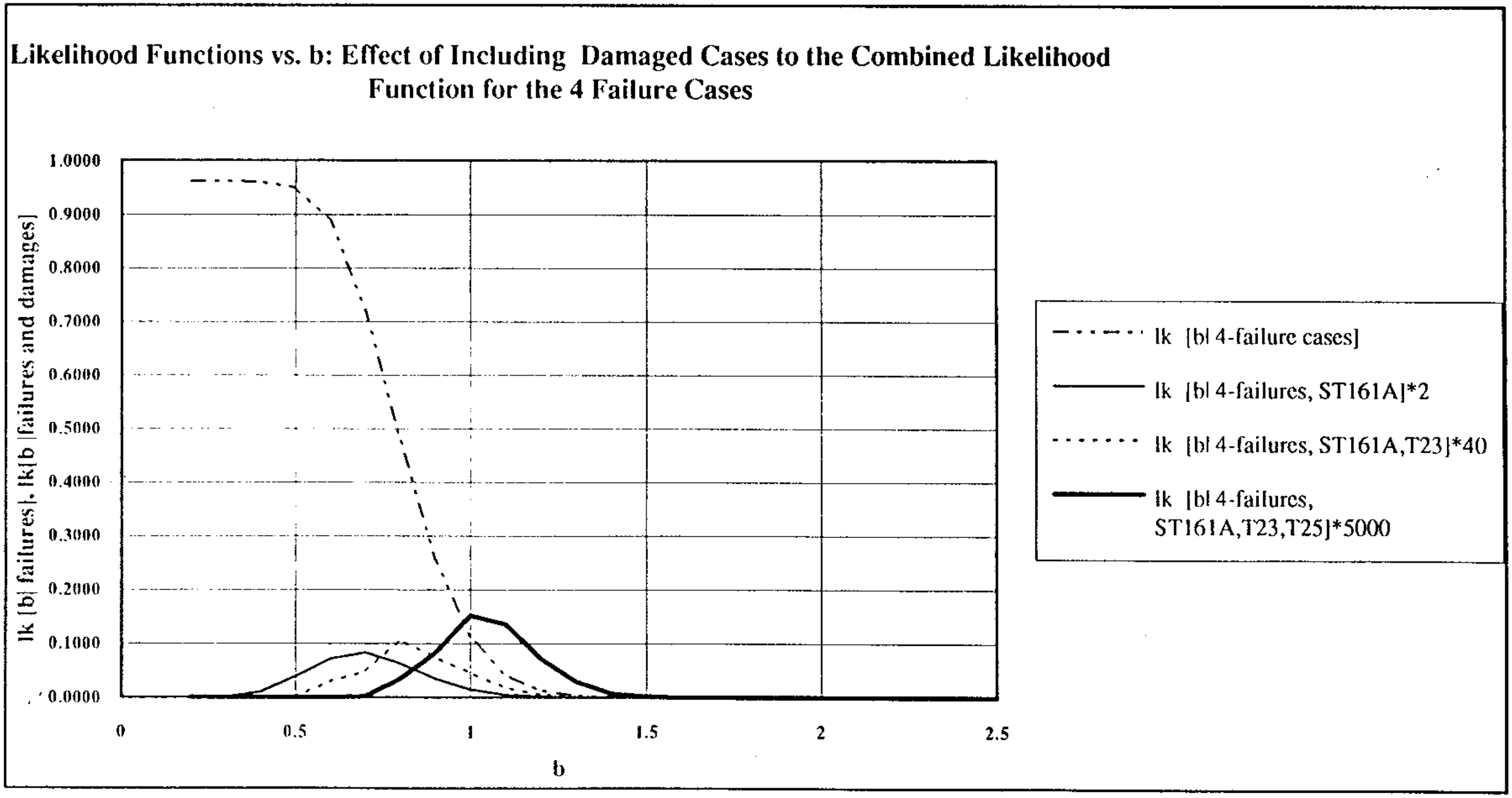


Figure 4-9 Combined Likelihood Functions - All Failure and Successive Damage Cases

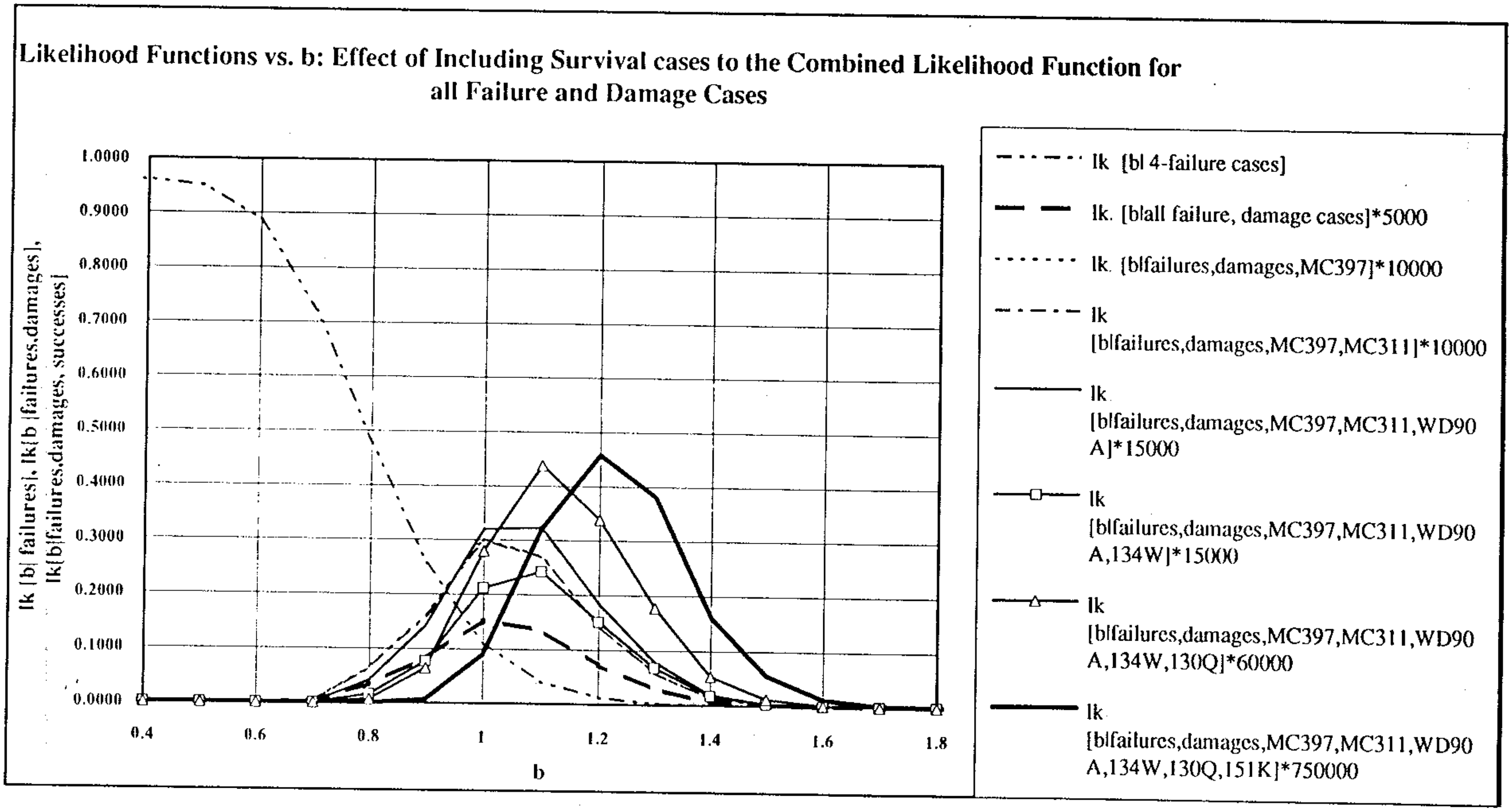


Figure 4-10 Combined Likelihood Functions - All Failure, Damage, and Successive Survival Cases

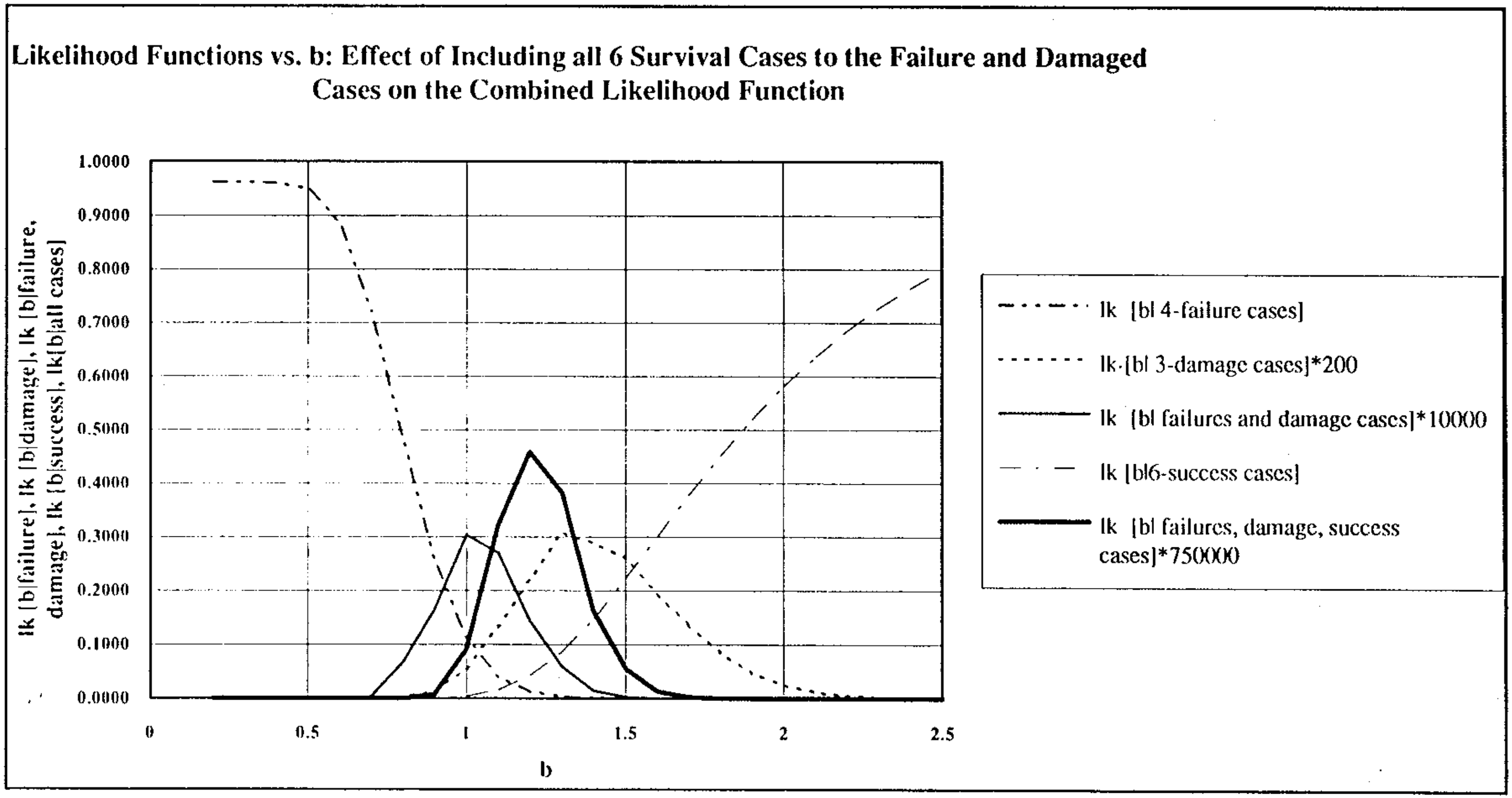


Figure 4-11 Combined Likelihood Functions - Different Categories and All Cases

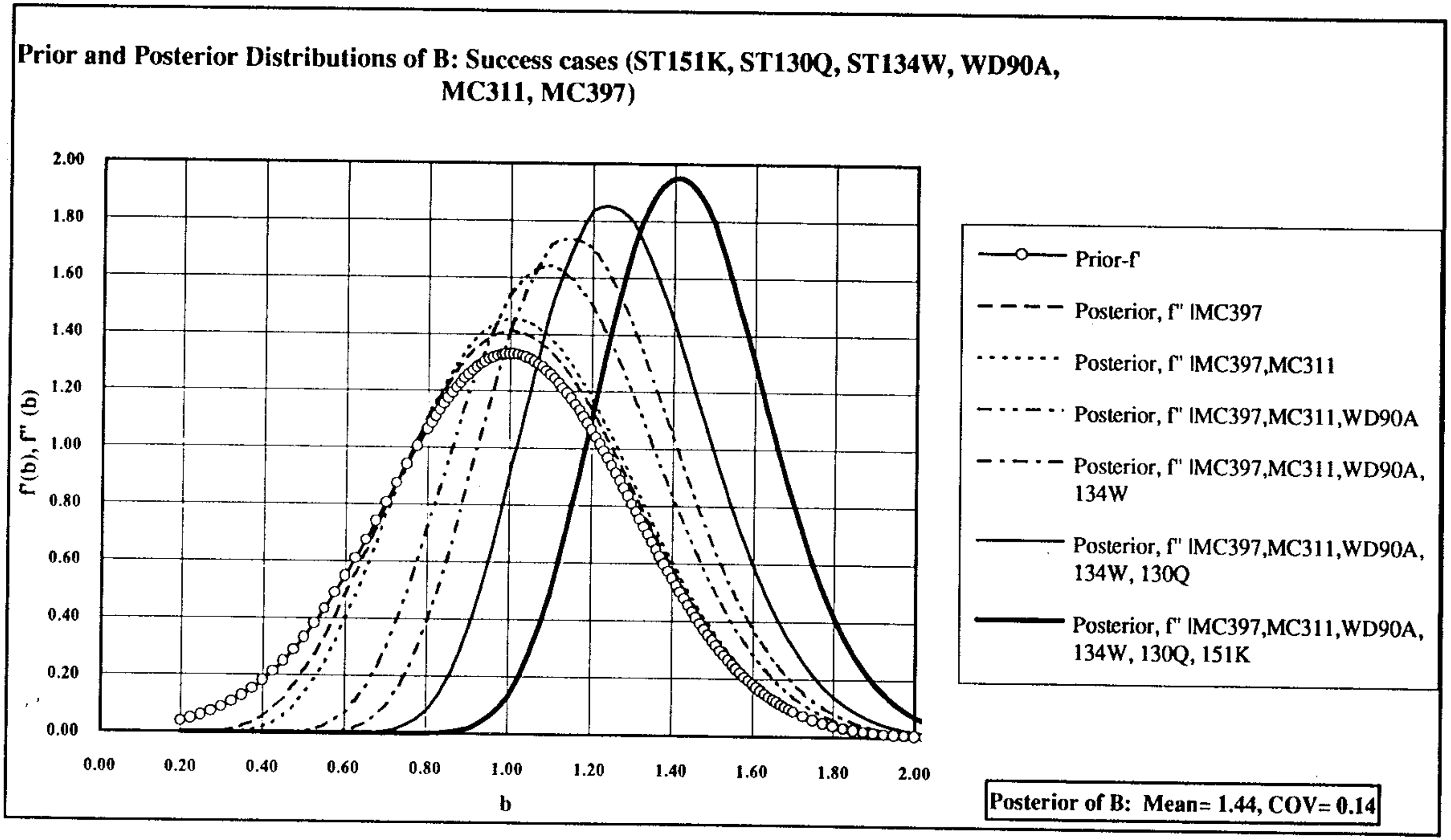


Figure 4-12 Posterior Distribution of Bias Factor (B) - Effect of Combination of Successive Success Cases

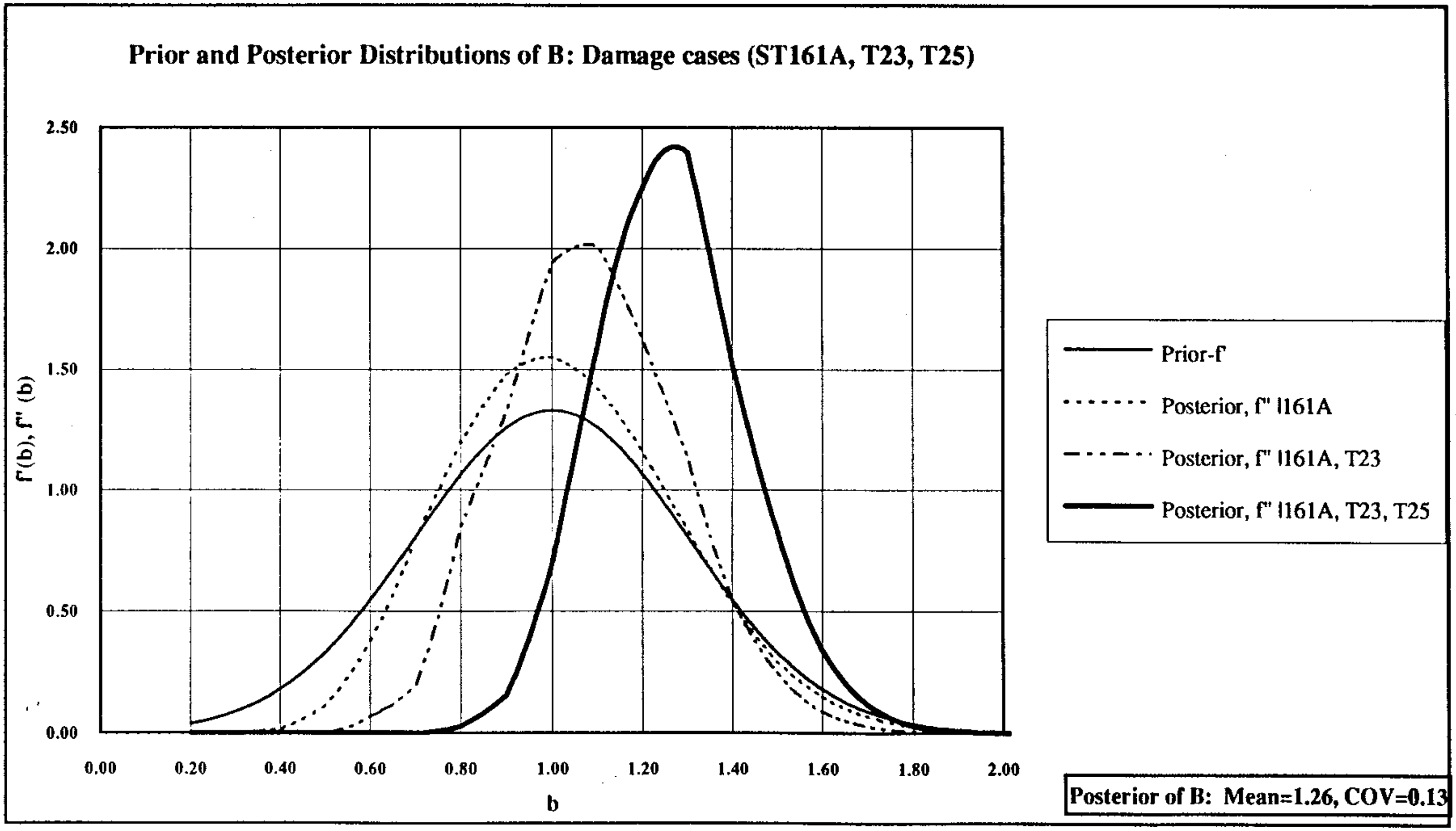


Figure 4-13 Posterior Distribution of Bias Factor (B) - Effect of Combination of Successive Damage Cases

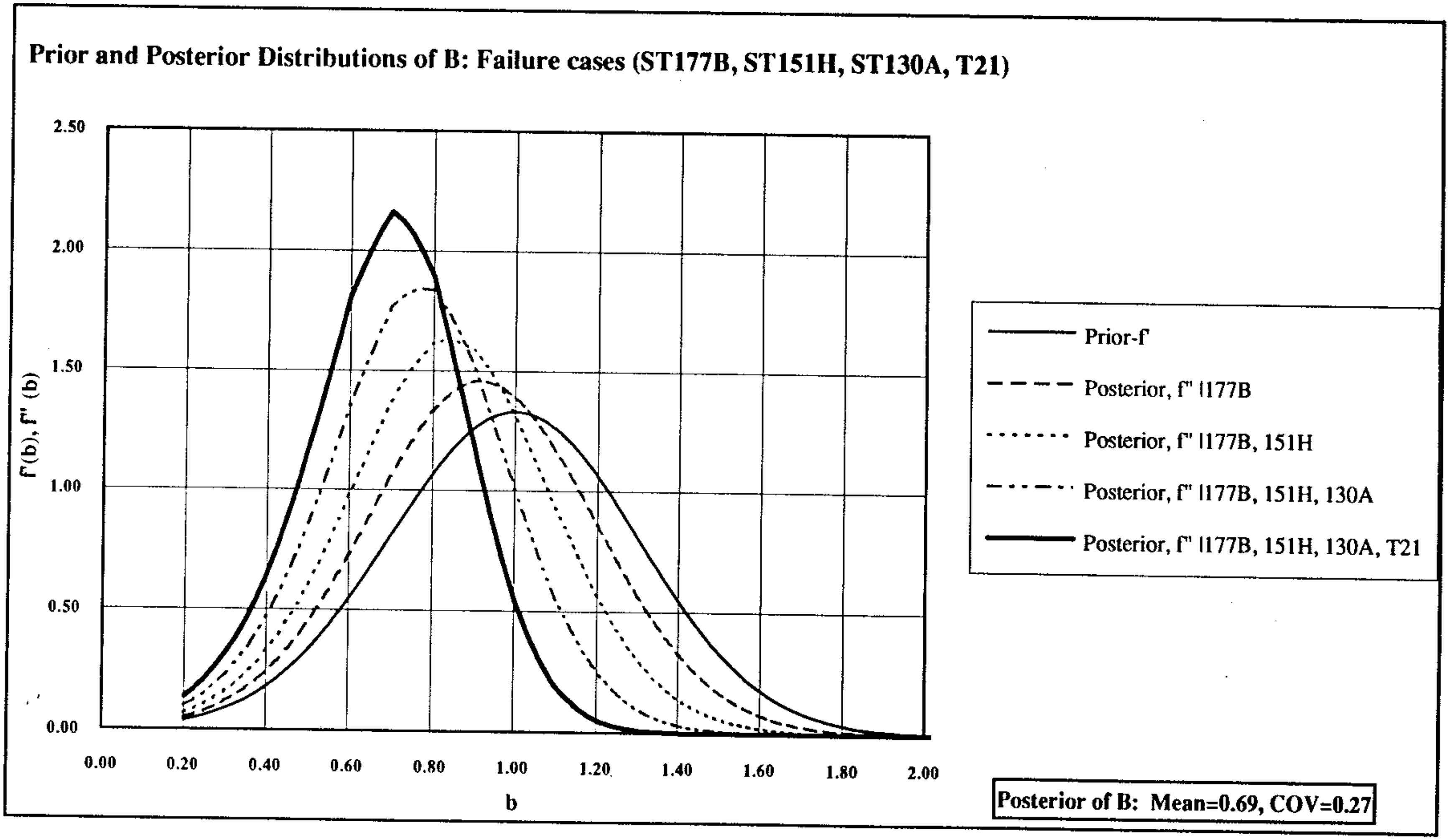


Figure 4-14 Posterior Distribution of Bias Factor (B) - Effect of Combination of Successive Failure Cases

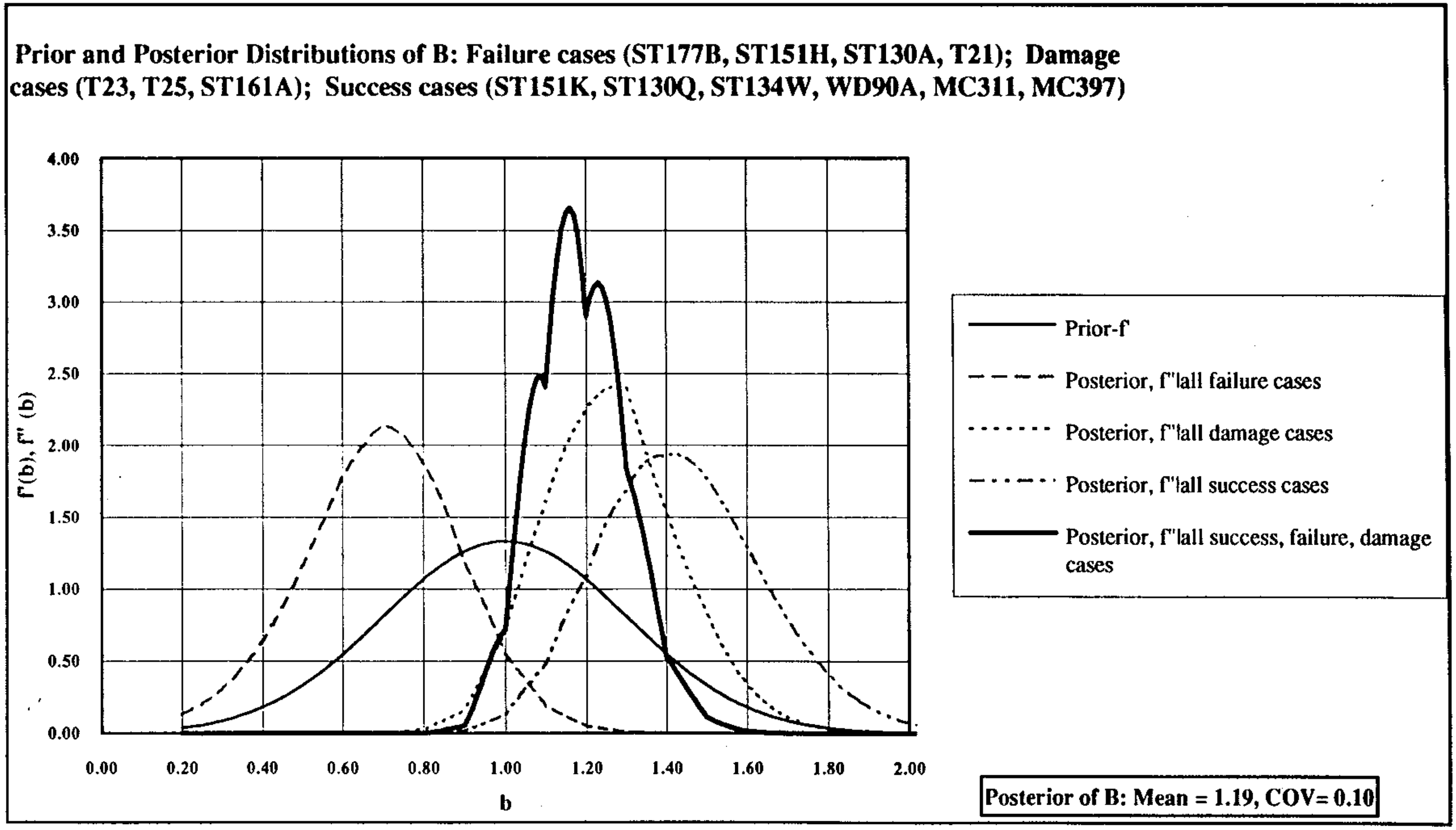


Figure 4-15 Posterior Distribution of Bias Factor (B) - Different Categories and All Cases Combined

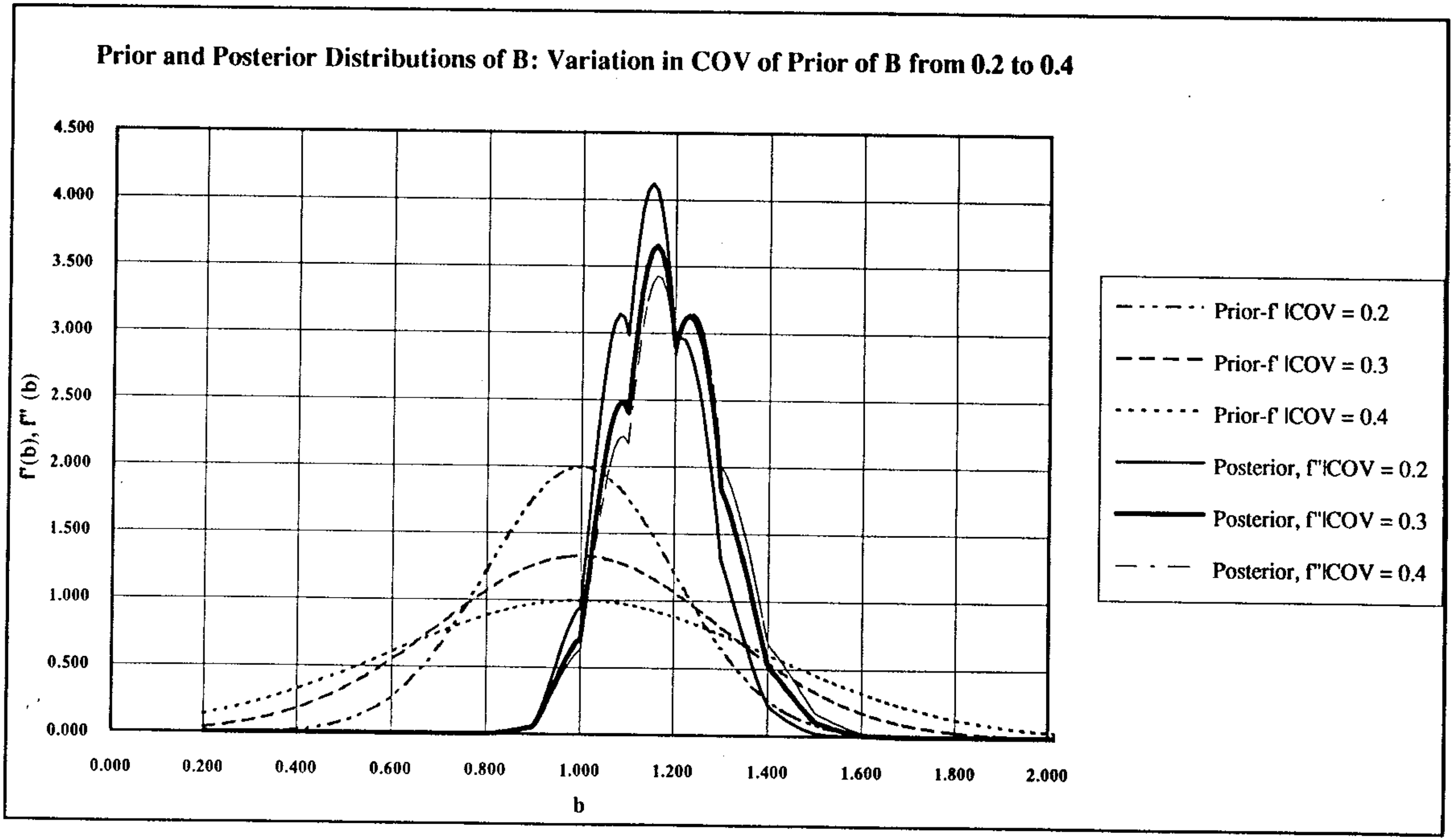


Figure 4-16 Posterior Distribution of Bias Factor (B) - Effect of Variation in COV of Prior of B

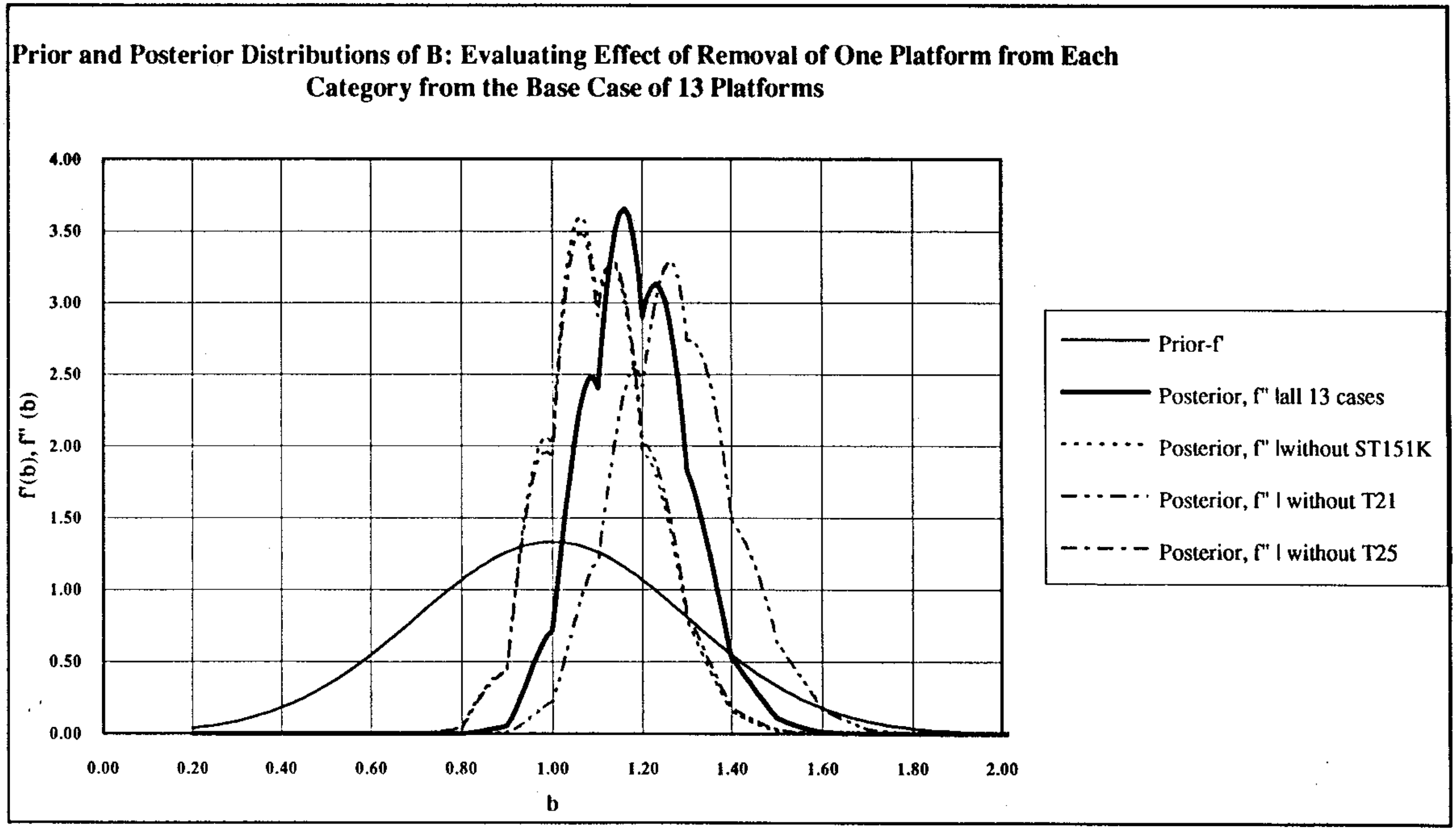


Figure 4-17 Posterior Distribution of Bias Factor (B) - Effect of Removal of 1-Platform from Any Category

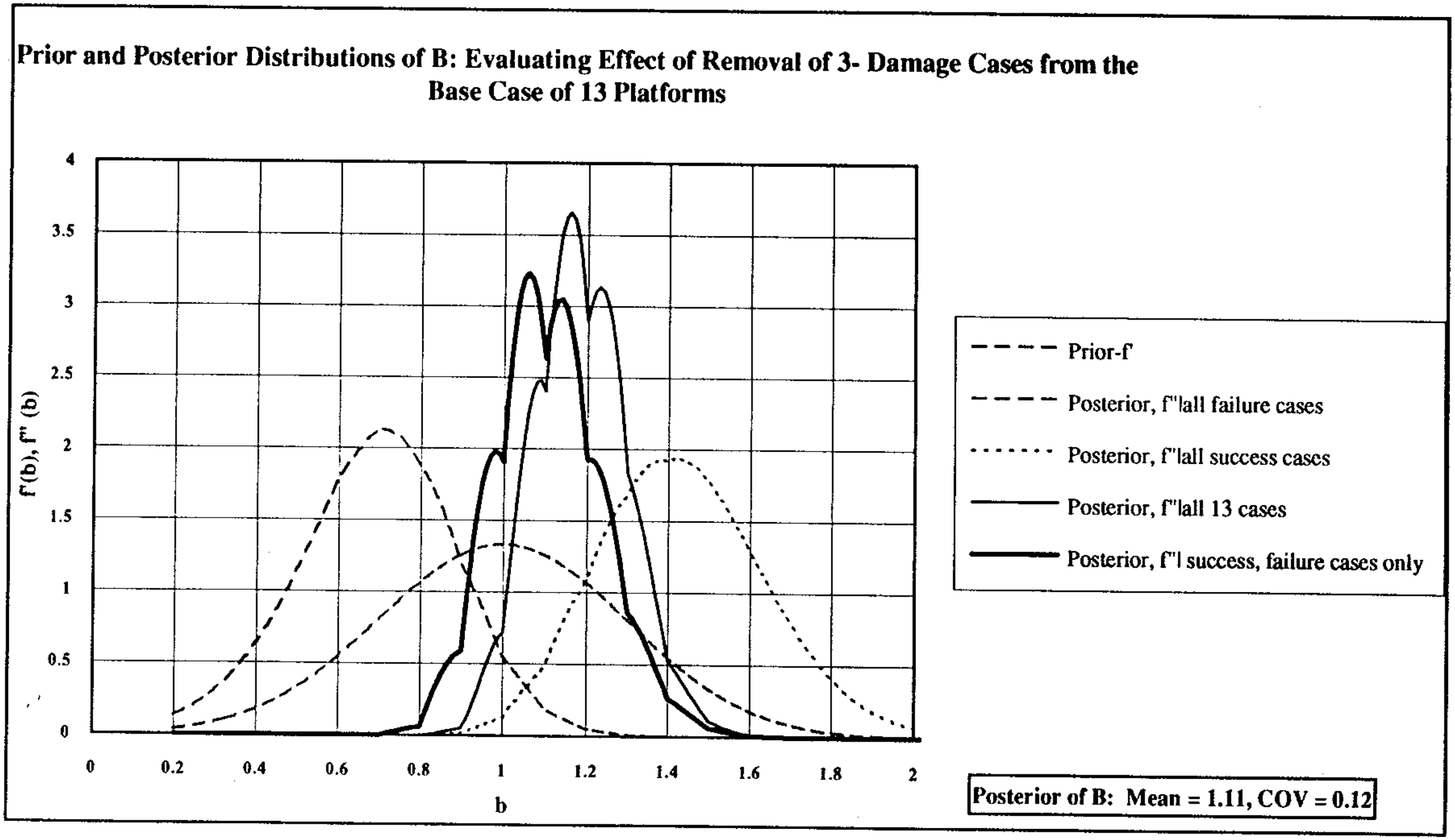


Figure 4-18 Posterior Distribution of Bias Factor (B) - Effect of Removal of 3-Damage Cases

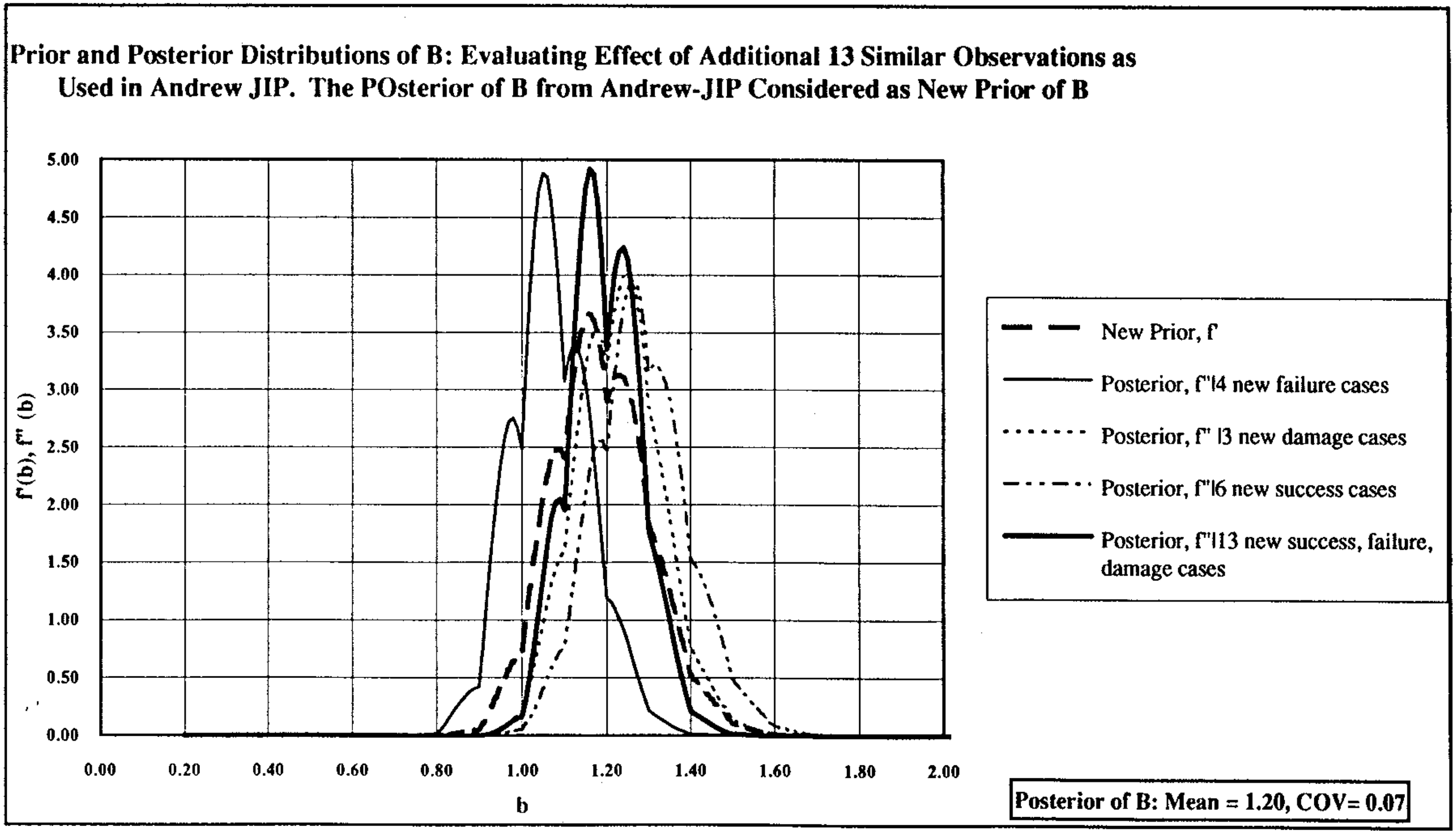


Figure 4-19 Posterior Distribution of Bias Factor (B) - Effect of Additional 13-Similar Observations

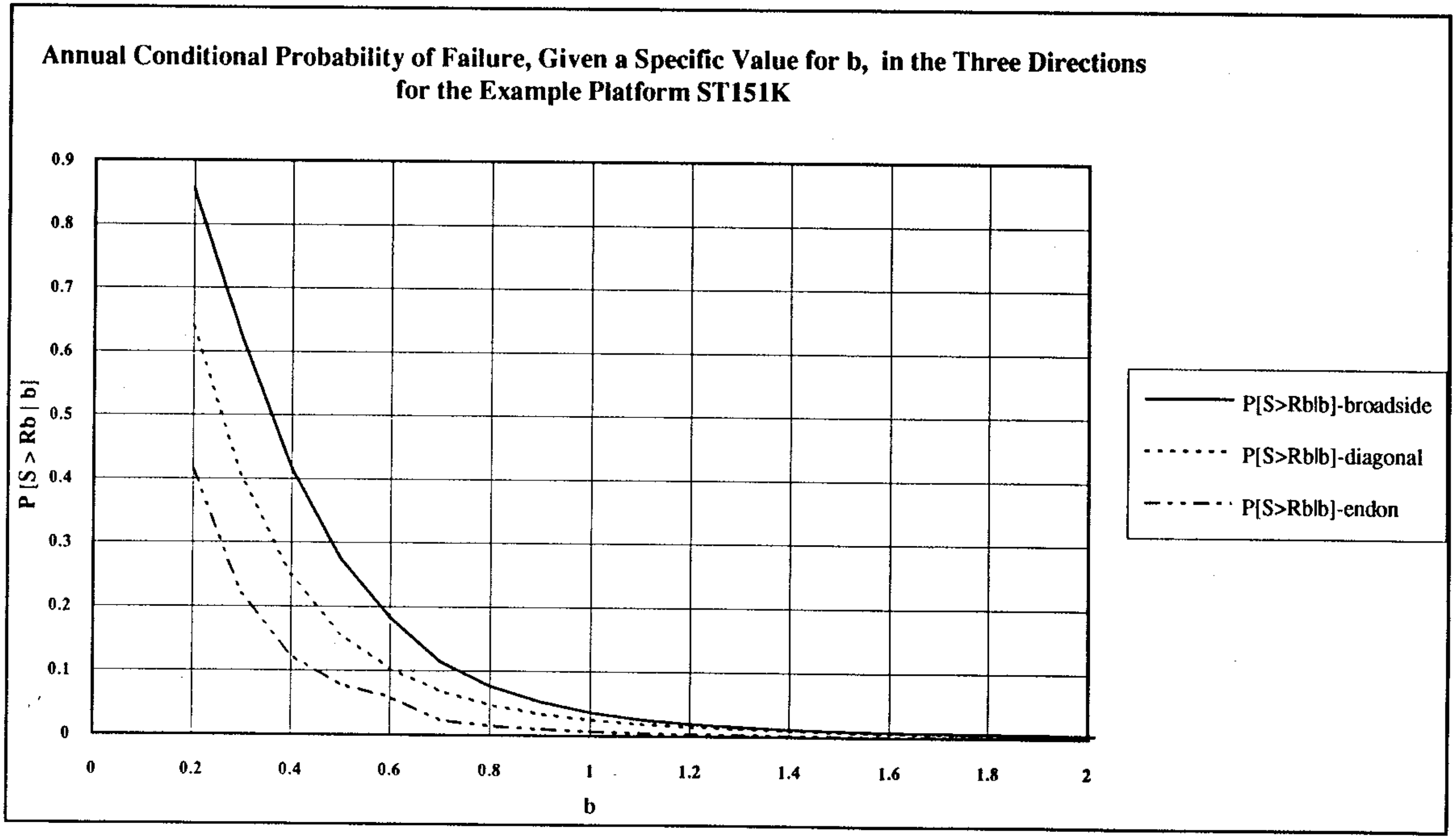


Figure 4-20 Variation in Conditional Annual Probability of Failure given b - Example Platform ST151K

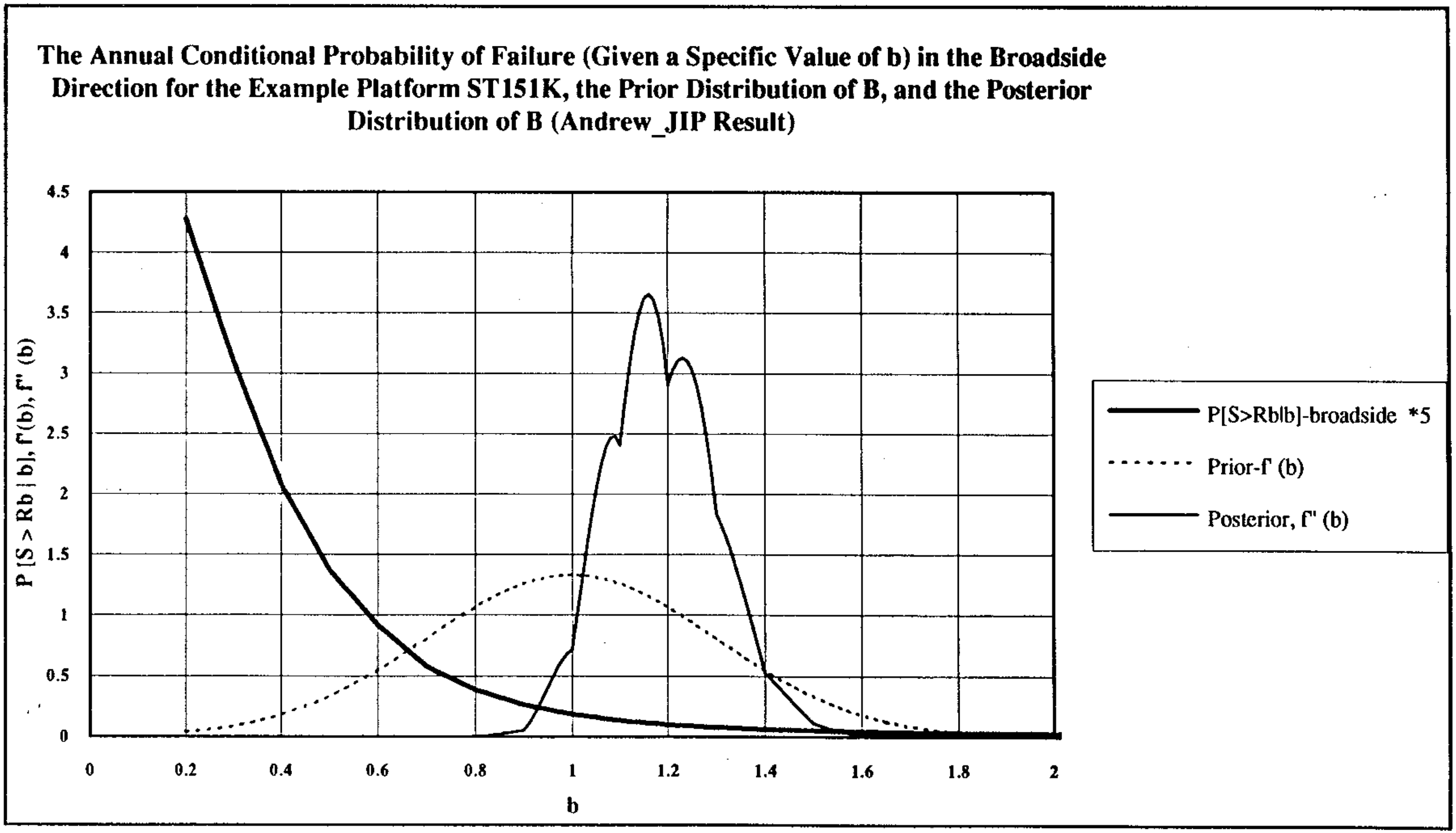


Figure 4-21 Prior and Posterior Distributions of B, and Conditional Annual Probability of Failure : Broadside Direction - Example Platform ST151K

93/07/23
11:30:19

Andrew_J.. _ST151K
broad1.inp

```
.ATFORM ST151K   BROADSIDE_Revised  DIR.
›efficients C1,C2,C3.  FROM CHEVRON DATA

.0          57.0          60.0          69.0          1000.0
    0.62900      0.0000017      0.02000      0.00000038
    6.54200      2.21900        3.91600      1.99500
    2.08500      5.27400        2.97400      5.55200
orm Data:  Number of intervals, hours per interval and hindcast data
 1.0
3.70   26.59  30.64  34.68   (Hs)
:.90    1.20   1.50   1.80   (U)
?.99    8.66   9.18  10.06   (Tz)
)V of eps0
.25
›dians, COV's:
?07.0  0.15          (Capacity)
  1.0  0.10          (Error in hindcast Hs)
  1.0  0.15          (Error in hindcast U)
.tegration: No. of sigmas for limits, and Number of Points
.07     3.40    15  (Base shear)
.00     2.00    11  (Error in hindcast Hs)
.00     2.00    11  (Error in hindcast U)
.252    2.00    51  106.86  (Individual wave height)
:der of polynomial for Gauss CDF

›rristal Distribution
.126   2.263
›timization Multipliers
.5 2.0
:lowed %ge change during paramteric study runs for diff. Pf's
No. of values
.001 0.01 0.1 1.0  Probab. of failures
?.0  6.00 2.0 1.0  Allowed %ge change
›timization Limits
.0 0.2        Factors for decreases
.0 5.0        Factors for increases
.0 0.001      Change ratios corresp. to above factors, decreasing
.0 1000.0     Change ratios corresp. to above factors, increasing
```

C1C2C3

STORM DATA

DISTRIBUTION PARAMETERS

INTEGRATION PARAMETERS

COMPUTATION CONTROL

Table 4-1 PF Program Input - Example

LATFORM ST151K BROADSIDE_Revised DIR.

NPUT VALUES
IMITS FOR INTEGRATION :

VARIABLE	--NO. OF SIGMAS-- LOW	HIGH	--ACTUAL VALUES---- LOW	HIGH	NO. OF POINTS
X	2.07	3.40	2794.	6118.	15
E1	2.00	2.00	0.804	1.206	11
E2	2.00	2.00	0.708	1.315	11
HT	1.25	2.00	0.00	56.39	51

IKELIHOOD FUNCTION GIVEN FAILURE

C	LIKELIHOOD
0.20000	0.98987
0.30000	0.98987
0.40000	0.98987
0.50000	0.98978
0.60000	0.98819
0.70000	0.97902

ATFORM ST151K BROADSIDE_Revised DIR.

LIKELIHOOD CURVE
FOR b= 0.2 to 0.7
(SEE INPUT TABLE 4-1)

PUT VALUES
MITS FOR INTEGRATION :

VARIABLE	--NO. OF SIGMAS-- LOW	HIGH	--ACTUAL VALUES---- LOW	HIGH	NO. OF POINTS
X	2.40	3.40	2593.	6118.	17
E1	2.55	2.00	0.749	1.206	13
E2	2.78	3.79	0.590	1.586	19
HT	1.25	4.06	0.00	92.11	227

KELIHOOD FUNCTION GIVEN FAILURE

C	LIKELIHOOD
0.70000	0.98294
0.80000	0.95391
0.90000	0.89987
1.00000	0.82233
1.10000	0.72968
1.20000	0.63236
1.30000	0.53892
1.40000	0.45456
1.50000	0.38142

TFORM ST151K BROADSIDE_Revised DIR.

LIKELIHOOD CURVE
FOR b= 0.7 to 1.5

INPUT VALUES
LIMITS FOR INTEGRATION :

VARIABLE	--NO. OF SIGMAS-- LOW	HIGH	--ACTUAL VALUES---- LOW	HIGH	NO. OF POINTS
X	2.40	3.40	2593.	6118.	17
E1	2.96	4.56	0.708	1.463	23
E2	3.92	4.13	0.417	1.638	25
HT	1.25	4.91	0.00	106.85	507

LIKELIHOOD FUNCTION GIVEN FAILURE

C	LIKELIHOOD
1.60000	0.31948
1.80000	0.22416
2.00000	0.15893
2.20000	0.11450
2.40000	0.08413

LIKELIHOOD CURVE
FOR b= 1.6 to 2.4

Table 4-2 PF Program Output - Example

AMOCO ST161A Broadside 43 to 62

```
INPUT DATA POINTS
  BASE SHEAR       HEIGHT        CURRENT
    1346.0          43.00          3.200
    1993.0          53.00          3.500
    2283.0          56.00          3.402
    2580.0          58.70          3.600
    2925.0          62.00          4.100
```

BASE SHEAR VALUES
FROM STRUCTURAL ANALYSIS

```
STARTING VALUES
        C1            C2            C3
        0.3          3.50         2.300
        0.2          3.50         2.300
        0.3          3.40         2.300
        0.3          3.50         2.200

TOLERANCE =     1.00000E-06
```

C1C2C3
SEED VALUES

```
FINAL RESULTS
PLEASE NOTE THAT THESE VALUES ARE UNITS-DEPENDENT
BEST FIT VALUES ARE ...
C1        0.217125
C2          1.46366
C3          2.25328
```

COMPUTED C1C2C3

```
ROOT MEAN SQUARE ERROR OF BEST FIT SOLUTION IS      33.6487

COMPARISON OF INPUT AND BEST FIT BASE SHEARS
     H          U     ........ BASE SHEAR.........
                       INPUT    BEST FIT     DIFF.
   43.00      3.200   1346.0    1313.8      -32.2
   53.00      3.500   1993.0    2052.4       59.4
   56.00      3.402   2283.0    2286.8        3.8
   58.70      3.600   2580.0    2547.2      -32.8
   62.00      4.100   2925.0    2923.3       -1.7
```

ERROR CHECK

Table 4-3 C1C2C3 Program Input and Output - Example

Table 4-4: Hindcast Seastate Data and Base Shear Variation with Storm Hour and Direction

Platform ST151K   Water Depth = 137 ft.

Hindcast Data							Base Shear Coeficients					
Storm Hour	Wave Direction	Hs	H max = Hs * 1.683	Peak Spectral T Tp	Zero Crossing Period Tz= 0.74* Tp	Current U	C1	C2	C3	H+C2*U	Expected Maximum Hindcast Base Shear	Remarks
	(Degree)	(ft.)	(ft.)	(sec.)	(sec.)	(knots)					(Kips)	
Broad-Side Direction (232.5 deg. to 277.5 deg.)												
1	251.9	18.11	30.476	9.94	7.35	0.60	0.301	5.99	2.24	34.049	813.74	
2	253.3	20.79	34.984	10.88	8.05	0.75	0.301	5.99	2.24	39.467	1132.73	
3	252.9	23.18	39.011	11.25	8.32	0.95	0.301	5.99	2.24	44.708	1497.75	
4	255.0	25.96	43.682	11.91	8.81	1.21	0.301	5.99	2.24	50.933	2005.62	
5	258.3	29.64	49.882	12.72	9.42	1.53	0.301	5.99	2.24	59.055	2793.76	
6	268.2	33.89	57.029	13.67	10.12	1.87	0.00356	3.60	3.38	63.748	4472.54	Wave in Deck
Diagonal Direction (277.5 deg. to 322.5 deg.)												
7	286.4	36.16	60.854	14.45	10.69	2.04	0.00287	4.10	3.38	69.230	4765.26	Wave in Deck
8	305.8	33.45	56.288	13.85	10.25	1.77	0.169	6.21	2.36	67.259	3478.36	Wave in Deck
9	314.2	31.06	52.277	12.95	9.58	1.34	0.169	6.21	2.36	60.623	2722.14	
10	317.7	28.97	48.762	12.25	9.06	0.92	0.169	6.21	2.36	54.472	2114.71	
11	321.6	27.50	46.278	11.76	8.70	0.52	0.169	6.21	2.36	49.522	1688.94	
End On Direction (322.5 deg. to 367.5 deg.)												
12	324.9	25.79	43.410	11.40	8.44	0.31	0.154	5.33	2.36	45.040	1230.32	
13	327.4	24.01	40.412	10.92	8.08	0.12	0.154	5.33	2.36	41.028	987.19	
14	329.2	22.54	37.927	10.67	7.90	0.02	0.154	5.33	2.36	38.009	824.25	
15	330.8	21.20	35.681	10.52	7.78	0.00	0.154	5.33	2.36	35.681	710.02	
16	331.7	19.98	33.620	10.25	7.58	0.00	0.154	5.33	2.36	33.620	617.02	
17	331.7	18.66	31.409	9.87	7.30	0.00	0.154	5.33	2.36	31.409	525.51	

Table 4-5: Summary of Physical Characteristics of Platforms

Platform Name	Water Depth ft.	Year of Installation	Physical Characteristics: Number of legs & batter	Leg-Pile annulus grouted/ ungrouted	Brace type in broadside frames	Brace type in endon frames	Sub-cellar deck elevation ft.	Cellar deck elevation ft.	Main deck elevation ft.
Survival Platform Cases:									
ST151K	137	1963	8-double	grouted	diagonal	K-braces	-	36.500	46.875
ST130Q	170	1964	4-double	grouted	K-braces	K-braces	-	39.000	50.500
ST134W	137	1981	4-double at 2legs	ungrouted	diagonal	diagonal	-	43.000	60.000
WD90A	184	1964	8-double	grouted	diag. & K	K-braces	26.750	34.833	50.104
MC311	343	1978	8-double	ungrouted	diagonal	X-braces	-	57.000	80.950
MC397	468	1991	4-double	ungrouted	X-braces	X-braces	53.500	65.000	93.000
Damage Platform Cases:									
T23ST52	63	1969	4-double	grouted	K-braces	K-braces	-	52.594	68.521
T25SS139	62	1969	4-double	grouted	K-braces	K-braces	-	50.594	67.240
ST161A	118	1964	8-double	grouted	diagonal	K-braces	-	34.750	49.945
Failure Platform Cases:									
ST177B	142	1965	8-double	grouted	diagonal	K-braces	-	39.000	50.500
ST151H	137	1964	8-single	grouted	diagonal	K-braces	-	39.500	48.625
ST130A	140	1958	8-single	grouted	diagonal	K-braces	-	55.000	67.000
T21ST72	61	1969	4-double	grouted	K-braces	K-braces	-	54.594	71.240

Table 4-6: Summary of Hindcast Maximum Seastate During Andrew at Platform Locations

Platform Name	Water Depth ft.	Year of Installation	Sea State Data: Hindcast maximum wave height H ft.	Hindcast maximum current U ft/sec	Hindcast maximum crest height Hc ft.	Extreme wave dir. from true North degree	1st and 2nd Direction Considered in Calibration	Remarks
Survival Platform Cases:								
ST151K	137	1963	60.854	3.443	40.198	286.40	Broadside & Diagonal	Wave in cellar deck
ST130Q	170	1964	62.269	3.000	41.341	289.10	Diagonal	Wave in cellar deck
ST134W	137	1981	60.854	3.443	40.198	286.40	Endon & Diagonal	-
WD90A	184	1964	50.531	2.560	33.457	303.36	Diagonal & Endon	Wave in sub-cellar deck
MC311	343	1978	62.000	2.384	40.000	288.30	Broadside & Diagonal	-
MC397	468	1991	65.911	1.330	40.727	273.40	Broadside & Diagonal	-
Damage Platform Cases:								
T23ST52	63	1969	50.300	4.130	31.382	286.37	Diagonal	-
T25SS139	62	1969	50.630	4.250	30.920	257.70	Orthogonal	-
ST161A	118	1964	58.495	3.560	40.228	290.40	Broadside	Wave in cellar deck
Failure Platform Cases:								
ST177B	142	1965	60.167	3.460	41.800	267.60	Diagonal-1	Wave in cellar deck
ST151H	137	1964	60.854	3.443	40.198	286.40	Diagonal	Wave in cellar deck
ST130A	140	1958	60.950	3.410	40.290	286.62	Diagonal	-
T21ST72	61	1969	49.740	4.260	31.072	257.31	Orthogonal	-

Table 4-7: Summary of Expected Hindcast Load and Capacity Levels, Probabilities of Failure, and Classifications for Platforms

Platform Name	Water Depth ft.	Year of Installation	1st and 2nd Direction Considered in Calibration	Direction -1 Ultimate Capacity Ru (kips)	Direction -1 Expected Max. Hindcast Base Shear S (kips) (Approx.)	Direction -1 Ratio Ru / S	Direction -2 Ultimate Capacity Ru (kips)	Direction -2 Expected Max. Hindcast Base Shear S (kips) (Approx.)	Direction -2 Ratio Ru / S	Results from PF Program (**): Probab. of failure in 1st direction at b=1.0	Probab. of occurrance of load level for damaged platforms	Probab. of failure in 2nd direction at b=1.0	Total probab. of survival in both dirns. at b = 1.0	Most Likely Classification of Platform
Survival Platform Cases:														
ST151K	137	1963	Broadside & Diagonal	3500	4473	0.78	3500	4765	0.73	0.78		0.88	0.03	Unexpected Survival
ST130Q	170	1964	Diagonal *	1265	1214	1.04				0.67			0.33	Unexpected Survival
ST134W	137	1981	Endon & Diagonal	1923	1307	1.47	1915	1118	1.71	0.25		0.12	0.66	Expected Survival
WD90A	184	1964	Diagonal & Endon	3130	1856	1.69	3267	2029	1.61	0.12		0.19	0.71	Expected Survival
MC311	343	1978	Broadside & Diagonal	20700	5606	3.69	17900	6382	2.80	0.00		0.01	0.99	Sure Survival
MC397	468	1991	Broadside & Diagonal	13718	4938	2.78	11566	3144	3.68	0.00		0.01	0.99	Sure Survival
Damage Platform Cases:														
T23ST52	63	1969	Diagonal: @ Ru	2006	1092	1.84				0.01				Expected to survive & likely to have damage
			@ 0.49 Ru	989	1092	0.91				0.73				
			@ 0.55 Ru	1105	1092	1.01				0.59				
			0.49 Ru < S < 0.55 Ru								0.15			
T25SS139	62	1969	Orthogonal: @ Ru	1342	1691	0.79				0.87				Unexpected to survive & most likely to have multiple damage
			@ 0.69 Ru	923	1691	0.55				0.99				
			@ 0.84 Ru	1132	1691	0.67				0.96				
			0.69 Ru < S < 0.84 Ru								0.03 **			
ST161A	118	1964	Broadside: @ Ru	4426	3973	1.11				0.51				Expected to survive & likely to have damage
			@ 0.91 Ru	4014	3973	1.01				0.58				
			0.91 Ru < S < 1.0 Ru								0.07			
Failure Platform Cases:														
ST177B	142	1965	Diagonal-1	4168	5150	0.81				0.77				Expected failure
ST151H	137	1964	Diagonal	3999	4206	0.95				0.69				Expected Failure
ST130A	140	1958	Diagonal	3000	2779	1.08				0.53				Likely Failure
T21ST72	61	1969	Orthogonal	1984	1615	1.23				0.40				Likely Failure ***

Notes :
- * Four leg platform with diagonal direction governing
- ** Probability at b = 1.8 is 0.31
- *** Classified as likely failure due to additional unknowns such as site specific soils and shallow water wave forces

Section 5
Conclusions and Recommendations

5.1 DATABASE

The database indicates that a majority of the severely damaged or failed platforms were of 1960's or earlier vintage with many having incurred wave-in-the-deck loads during Andrew. Only three platforms installed after 1977 (API RP 2A, 9th edition) were damaged or failed. Post storm investigation indicates that these platforms were likely affected by other than large waves (e.g., pre-existing damage). This is an important observation since API is currently considering design by the 9th edition or later of API RP 2A as one of the criteria for reassessment.

The database was not as extensive as originally anticipated due to lack of data from non-participants. However, the variety of survived, damaged and failed platforms available from participants provided a good sampling of platform consequences for the calibration. The primary recommended further work for the database includes gathering additional information on failures/survivals to support additional calibrations (see below).

5.2 CAPACITY ASSESSMENTS

In some cases, significant differences were observed between the failed members identified by analysis and those observed following Andrew. Further and more detailed analysis may be needed to establish which elements of the design recipe lead to such differences in results. Based on the analysis performed in this project, the following interpretations are made:

- **K-Joints.** For many of the old platforms, K-joints have been found to govern failure of first component and/or successive failures leading to collapse. However, the formula used to determine the capacity of the K-joints (API RP 2A) gives a lower bound estimate. In addition, the joint capacity has been modeled by an elastic-plastic non-load shedding truss element. A load-shedding strut-type element may be more appropriate for these older structures. Further structural analysis using capacity formulations and different joint modeling procedures may lead to different results.

- **Brace Capacity.** The braces in the vertical frames are modeled as struts. The capacity of the strut is dependent on the effective length factor (K) and lateral load on the members. In the present work, the effective length factor has been taken as 0.65. This value of K has been used irrespective of brace type (K, diagonal, X), size of chord members, or grouting of leg-pile annulus. Actual end-fixity of braces may vary the effective length factor from the 0.65 condition used in this study.

- **Foundations.** The observed behavior indicated no pile failures, whereas the analysis indicated that pile failures may have occurred in some cases. Typical soil properties and strength variation with depth were used for a number of platforms analyzed for this study. Actual properties at some of the platform sites may vary. Further analysis using different soil strengths and stiffnesses may produce results closer to observed foundation performance.

It is recommended that further investigation of the above topics be considered as follow-on work to this project.

5.3 CALIBRATION WORK

A factor known either as a bias, calibration or correction factor has been developed to represent modeling uncertainties with respect to the overall safety factor (resistance divided by loading effects) for platforms during extreme hurricane loadings.

The final posterior of the bias factor (B) had a mean value of 1.19 with an uncertainty (measured as the COV) of 10 percent. This implies that on average, for the platforms evaluated by this project, there is about a 19% conservatism in the assessment "recipe" used by the project. Its application to an example platform showed the posterior probability of failure reduces due to increased median and small COV of the bias factor. The small COV (0.10) of the bias factor has little effect compared to the random variabilities, e.g., in the annual H_s value or in the C_D coefficient.

The favorable outcome for the bias factor, (i.e., "B" > 1) indicates that the current platform checking process is conservative in the sense that more failures are predicted during storms than will actually occur.

Applying the bias factor to a single specific structure for the purposes of "requalification," however, may lead to erroneous conclusions. The bias factor was averaged from a fleet of structures that were exposed to hurricane Andrew. These structures had a variety of potential platform failure modes with varying degrees of criticality. These included some subjectivity in interpreting the platform safety margins at specific locations. The exposed platforms may not be representative of any specific structure about which detailed predicted model capacities and safety margins are available. Further studies (as recommended below) should be able to refine the bias factors for specific platforms conditions and failure mode types.

The direct application of the bias factor is justified in performing economic risk/cost/benefit remediation studies for fleets of older platforms. The favorable bias factor will therefore increase the average platform reliability to resist hurricanes beyond that computed by conventional analysis. Any global remediation decisions may be based on the updated reliabilities, which reflect the observed Andrew and other hurricane experiences.

The updated bias factor and the associated improved reliability estimates are potentially applicable also to the development of the new API requalification criteria. The application is legitimate for those cases when the criteria are explicitly based on a target risk level such as, for example, an annual failure rate of one per thousand. When the acceptance criteria are established by direct calibration with experience, however, the bias factor is implicitly reflected in such experience and cannot provide any further direct adjustments.

The maximum wave height for the shallow water platforms (T21, T23, and T25) is limited due to the wave breaking phenomenon. Recent research has indicated that the breaking wave height for shallow and intermediate water depths may differ from the (0.78 *d) used in this project [Tucker, 1991]. This issue may need further investigation for shallow water platforms.

The correlation between seastates, load level, and capacities in different directions has not been considered in this work. In further work the correlations may be considered to evaluate their effect on the bias factor.

Based upon the above, recommended topics for further study include:

- Develop "multiple" bias factors – for example, a bias factor for platforms governed by brace failure and a bias factor for platforms governed by foundation failure. This project developed a "global" bias factor irrespective of failure modes.

- Develop and implement a process that uses a "weighting" procedure that accounts for the performance of other Gulf of Mexico platforms during Andrew that were not directly evaluated by the project. The intent is to increase in a simplified manner the number of platforms used to establish the bias factor.

- Investigate more directly component damage (braces, legs, joints, etc.) predicted analytically versus component damage actually observed. This project focused primarily on the platform capacity for use in calibration.

- Investigate more thoroughly the results of the nonlinear analysis and, where necessary, re-perform some of the structural analyses used in the calibration based on an updated recipe (i.e. different joint capacity equation).

- Investigate platforms in different categories (i.e., configuration, water depth), which were affected by past hurricanes, e.g., Hilda, Betsy, Camille, etc. Almost all damaged platforms past Andrew were in shallow water depth limit (< 150 ft.) located essentially in South Timbalier and Ship Shoal blocks, and a majority of them had K-joint capacity as a key failure mode. Prior to Andrew, platform failures due to hurricanes were also reported in intermediate water depths up to 327 ft. Therefore, for a more representative sampling, it would be useful to include platforms which were hit by major hurricanes in different regions of the Gulf of Mexico with different structural characteristics, failure modes and mechanisms. Such an attempt combined with the weighting procedure would provide more representative multiple bias factors.

Section 6
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Appendix A
Database Hardcopy

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: South Timbalier 177 - Platform B Aux
Operator Name	: Chevron
Damage Summary	: Severely Damaged, to be Removed

Platform Information

Platform Type	:
Water Depth (ft)	: 140
Number of Piles	: 8
Number of Wells	:
Installation Date	: 1965
Design Criteria	: Unknown
Deck Elevation	:

Andrew Conditions

Hindcast Hs	: 11 Meters
Distance from eye	: 6 Miles

Comments

- Platform similar to ST177 B and ST151 K
- Marine growth popped away
- Cracking in x bracing at waterline, South side (Row 1)'
- Joint failures at K bracing
- Platform shakes when waves pass by
- Movement of hatches indicating waves were large
- Severe damage--too expensive to repair
- Platform similar to ST177 B and ST 151 K used in calibration

Table A-1 Qualitative Case Study - 1

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: Mississippi Canyon 311 - Platform A (Bourbon)
Operator Name	: Shell
Damage Summary	: Survived

Platform Information

Platform Type	:
Water Depth (ft)	: 425
Number of Piles	:
Number of Wells	:
Installation Date	: 1978
Design Criteria	: Unknown
Deck Elevation	: Lower deck elev. +51'-0" T.O.S.

Andrew Conditions

Hindcast Hs	: 11 Meters
Distance from eye	: 32 Miles

Comments

- 8 to 10 sheets of grating torn up on lower deck around conductors (elev +51'-0")
- 90% of PVC drain piping suspended from lower deck gone (elev. +47'-0")
- Cage around ladder from +51' down to oil and water sump flattened right below lower deck at elevation +47'
- Sump landing grating missing and steel pipe knocked at bottom of oil and water sump severed
- Grating missing at +12' elevation
- Damage to underdeck piping indicates very large waves

Table A-2 Qualitative Case Study - 2

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: Ewing Bank 826, Platform A
Operator Name	: British Petroleum Exploration
Damage Summary	: Survived

Platform Information

Platform Type	: Self-Contained
Water Depth (ft)	: 488
Number of Piles	: 8 Legs with 12 Skirt Piles (8 exterior, 4 interior)
Number of Wells	: 31 of 48 conductors installed
Installation Date	: 1988
Design Criteria	: API RP 2A - 18 ed.
Deck Elevation	: +68'-0" (Production)

Andrew Conditions

Hindcast Hs	: 12 Meters
Distance from Eye	: 11 Miles

Comments

- One of the deepest water platforms exposed to large waves and winds during Andrew
- Platform design based upon newer API recommendations

Table A-3 Qualitative Case Study - 3

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: South Timbalier 188 - Platform CA
Operator Name	: Chevron
Damage Summary	: Deck torn off

Platform Information

Platform Type	:
Water Depth (ft)	: 143
Number of Piles	:
Number of Wells	:
Installation Date	: 1981
Design Criteria	: Unknown
Deck Elevation	: Lower deck is 62 feet from bottom of steel to waterline

Andrew Conditions

Hindcast Hs	: 11 Meters
Distance from Eye	: 5 Miles

Comments

- Deck Torn off by wind and found 200 feet away
- Appears that there was a tensile failure on lightly loaded side and bending failure on cantelever side
- No damage to jacket
- Deck was cantelever type

Table A-4 Qualitative Case Study - 4

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: South Timbalier 152 - Platform E
Operator Name	: Chevron
Damage Summary	: Collapsed

Platform Information

Platform Type	:
Water Depth (ft)	: 137
Number of Piles	: 8
Number of Wells	:
Installation Date	: 1960
Design Criteria	: Unknown
Deck Elevation	: +49'-0"

Andrew Conditions

Hindcast Hs	: 11 Meters
Distance from Eye	: 14 Miles

Comments

- Platform lying in bottom
- Appears as though platform destructed in place--deck in upright position on top of debris
- Platform had previously been strengthened with tripod reinforcements

Table A-5 Qualitative Case Study - 5

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: Ship Shoal 72 - Platform A
Operator Name	: Mobil
Damage Summary	: Collapsed

Platform Information

Platform Type	: Self-Contained gas gathering facility
Water Depth (ft)	: 29
Number of Piles	:
Number of Wells	:
Installation Date	: 1948
Design Criteria	:
Deck Elevation	:

Andrew Conditions

Hindcast Hs	: 6 Meters
Distance from Eye	: 8 Miles

Comments

- Platform collapsed, pipeline fire
- All wells P&A five years ago
- Estimated wave height at least 8 feet above deck
- Deck broke off
- Jacket broke off 2 feet above mudline
- One of oldest platforms in Gulf of Mexico

Table A-6 Qualitative Case Study - 6

Andrew Failure Consequences Database
Individual Case Report

Platform Name : South Timbalier 172 - Platform A

Operator Name : Samedan

Damage Summary : Damaged - to be salvaged

Platform Information

Platform Type	:
Water Depth (ft)	: 107
Number of Piles	: 6
Number of Wells	:
Installation Date	: 1964
Design Criteria	:
Deck Elevation	: Sub-cellar @ 29'-0", Cellar @ 36'-6"

Andrew Conditions

Hindcast Hs	: 10 Meters
Distance from Eye	: 1 Mile

Comments

- Structure leaning 20 degrees
- South East pile puled out 15 feet
- Adjacent east pile pulled out 5 feet
- North East and adjacent east deck leg sheared bellow cellar deck
- Remaining deck legs are bent and/or partially collapsed
- 360 degree tear in NE jacket leg (-10')
- 360 degree tear and collapse of north X-brace
- Following Andrew, deck removed 9/92, jacket removed 2/93

Table A-7 Qualitative Case Study - 7

Andrew Failure Consequences Database
Individual Case Report

Platform Name	: Ship Shoal 215 - Platform C
Operator Name	: UNOCAL
Damage Summary	: Collapsed

Platform Information

Platform Type	:
Water Depth (ft)	: 100
Number of Piles	: 4
Number of Wells	: 4
Installation Date	: 1962
Design Criteria	: Unknown
Deck Elevation	: Sub-cellar @ 34'-0", Cellar @ 42'-0", Prod @ 57'-0"

Andrew Conditions

Hindcast Hs	: 8 Meters
Distance from Eye	: 14 Miles

Comments

- Deck separated and fell to sea floor 200 feet away.
- Platform was damaged in Hilda in 1964. Repairs were made, however, cracks propagated from Hilda damage such that some jacket legs had cracks two feet and longer
- Platform was scheduled for removal in Sept. 1992.
- Damage prior to Andrew likely contributed to failure

Table A-8 Qualitative Case Study - 8

Notes:

1. The list is for MMS regulated waters only.

2. The list was compiled by PMB based upon available public information. The list was provided to participants for their input in the event that some platforms were inadvertently excluded. Participants were asked to provide details of any missing platforms or inaccurate information. The list included here is the updated version based on feedback from several participants. Since some participants did not provide feedback, there may still be some missing platforms.

Table A-9 Platforms in Path of Andrew Which Survived (Without Damage)

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
ADOBE RESOURCES CORPORATION	SS 0299	A	90	258	56	7
AEDC USA INC	MC 0486	A	90	582	42	13
AGIP PETROLEUM CO INC.	WD 0089	A	89	220	23	9
AGIP PETROLEUM CO INC.	WD 0089	C	69	220	23	9
ALLIANCE OPERATING CORPORATION	EI 0196	B	69	98	48	6
ALLIANCE OPERATING CORPORATION	EI 0196	C	73	96	48	6
ALLIANCE OPERATING CORPORATION	EI 0196	C-PROD	73	96	48	6
ALLIANCE OPERATING CORPORATION	EI 0204	G	75	112	51	6
ALLIANCE OPERATING CORPORATION	EI 0205	D	73	105	51	6
ALLIANCE OPERATING CORPORATION	EI 0205	F	76	105	51	6
ALLIANCE OPERATING CORPORATION	EI 0206	A	70	100	51	6
ALLIANCE OPERATING CORPORATION	EI 0206	A-PROD	71	100	51	6
AMERADA HESS CORPORATION	ST 0205	B	90	161	40	11
AMERADA HESS CORPORATION	ST 0206	A	90	173	44	11
AMERADA HESS CORPORATION	ST 0224	D	91	165	43	11
AMERADA HESS CORPORATION	ST 0225	C PLAT.	90	186	43	12
AMERADA HESS CORPORATION	WD 0034	A	80	35	10	6
AMOCO PRODUCTION COMPANY	EI 0193	A	59	87	44	6
AMOCO PRODUCTION COMPANY	EI 0196	H	81	93	48	6
AMOCO PRODUCTION COMPANY	EI 0215	12 PLATFORM	88	100	37	6
AMOCO PRODUCTION COMPANY	EI 0215	B	68	103	37	6
AMOCO PRODUCTION COMPANY	EI 0215	B-PROD	72	103	37	6
AMOCO PRODUCTION COMPANY	EI 0215	C	73	103	37	6
AMOCO PRODUCTION COMPANY	EI 0273	B	70	184	68	6
AMOCO PRODUCTION COMPANY	EI 0273	C	82	184	68	6
AMOCO PRODUCTION COMPANY	EI 0273	C-PROD	82	184	68	6
AMOCO PRODUCTION COMPANY	EI 0300	A	81	198	66	7
AMOCO PRODUCTION COMPANY	PL 0001	A	80	30	3	6
AMOCO PRODUCTION COMPANY	SS 0084	A	79	15	17	6
AMOCO PRODUCTION COMPANY	SS 0084	A-PROD	80	15	17	6
AMOCO PRODUCTION COMPANY	SS 0177	A	81	76	42	7
AMOCO PRODUCTION COMPANY	SS 0219	B	70	117	47	8
AMOCO PRODUCTION COMPANY	ST 0156	A	78	174	36	11
AMOCO PRODUCTION COMPANY	ST 0160	E	88	122	32	11
AMOCO PRODUCTION COMPANY	ST 0161	B	69	120	32	10
AMOCO PRODUCTION COMPANY	ST 0161	C	73	116	32	10
AMOCO PRODUCTION COMPANY	ST 0161	D	79	120	32	10
AMOCO PRODUCTION COMPANY	WD 0035	A	68	66	11	6
AMOCO PRODUCTION COMPANY	WD 0035	B	71	70	11	6
AMOCO PRODUCTION COMPANY	WD 0035	C	73	66	11	6
AMOCO PRODUCTION COMPANY	WD 0075	D	64	172	20	8
AMOCO PRODUCTION COMPANY	WD 0075	F	68	200	20	8
AMOCO PRODUCTION COMPANY	WD 0075	G	86	190	20	8
AMOCO PRODUCTION COMPANY	WD 0090	A	64	190	23	9
AMOCO PRODUCTION COMPANY	WD 0090	B	68	190	23	9
AMOCO PRODUCTION COMPANY	WD 0090	E	73	190	23	9
AMOCO PRODUCTION COMPANY	WD 0140	A	90	303	23	11
ARAN ENERGY CORPORATION	SP 0037	A	63	142	7	7
ARAN ENERGY CORPORATION	SP 0037	B	63	140	7	7
ATLANTIC RICHFIELD COMPANY	MC 0148	A	80	651	25	9
ATLANTIC RICHFIELD COMPANY	SP 0052	A	91	531	9	8
ATLANTIC RICHFIELD COMPANY	SS 0091	A	81	35	8	7
ATLANTIC RICHFIELD COMPANY	SS 0091	B	82	35	8	7
ATLANTIC RICHFIELD COMPANY	SS 0178	A	85	85	40	7
ATLANTIC RICHFIELD COMPANY	SS 0332	A	85	420	39	9
ATLANTIC RICHFIELD COMPANY	ST 0245	A	87	185	59	10
B T OPERATING CO	EI 0294	A	80	204	73	6
BP Exploration Inc.	EI 0315	A	82	240	79	6
BP Exploration Inc.	EW 0826	A	88	483	54	12
BP Exploration Inc.	MC 0020	A	84	475	19	8
BP Exploration Inc.	MC 0109	A	91	1030	16	9
CANADIANOXY OFFSHORE PRODUCTIO	EI 0257	C	72	152	56	7

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)						
ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
CANADIANOXY OFFSHORE PRODUCTIO	EI 0257	D	72	155	56	7
CANADIANOXY OFFSHORE PRODUCTIO	EI 0257	E	80	156	56	7
CANADIANOXY OFFSHORE PRODUCTIO	EI 0258	A	70	158	57	7
CANADIANOXY OFFSHORE PRODUCTIO	EI 0258	B	71	158	57	7
CENTURY OFFSHORE MANAGEMENT	EI 0298	A	89	211	62	7
CENTURY OFFSHORE MANAGEMENT	SS 0062	A	91	26	14	6
CENTURY OFFSHORE MANAGEMENT	ST 0107	A	90	79	25	10
CENTURY OFFSHORE MANAGEMENT	ST 0148	A	70	105	29	10
CHEVRON USA INC	EI 0215	B	72	102	37	6
CHEVRON USA INC	EI 0215	B-PROD	73	102	37	6
CHEVRON USA INC	EI 0215	C	74	100	37	6
CHEVRON USA INC	EI 0215	D	82	100	37	6
CHEVRON USA INC	EI 0229	A	90	123	46	6
CHEVRON USA INC	EI 0229	B	91	123	47	6
CHEVRON USA INC	EI 0230	CC	76	123	40	7
CHEVRON USA INC	EI 0230	CD	90	123	46	7
CHEVRON USA INC	EI 0231	A-PROD	70	111	39	7
CHEVRON USA INC	EI 0231	CA	68	111	39	7
CHEVRON USA INC	EI 0231	CB	71	106	39	7
CHEVRON USA INC	EI 0237	J	83	145	42	7
CHEVRON USA INC	EI 0238	A	63	139	42	7
CHEVRON USA INC	EI 0238	E	66	127	42	7
CHEVRON USA INC	EI 0238	F	72	145	42	7
CHEVRON USA INC	EI 0238	H	78	145	42	7
CHEVRON USA INC	EI 0252	B	65	148	49	7
CHEVRON USA INC	EI 0252	C	65	148	49	7
CHEVRON USA INC	EI 0252	G	77	148	49	7
CHEVRON USA INC	EI 0252	I	82	150	49	7
CHEVRON USA INC	EI 0305	A	75	221	73	6
CHEVRON USA INC	EI 0305	B	76	221	73	6
CHEVRON USA INC	EI 0341	A	78	272	81	6
CHEVRON USA INC	EI 0361	A	81	306	98	6
CHEVRON USA INC	GI 0026	BB	61	47	4	7
CHEVRON USA INC	GI 0026	P	56	40	4	7
CHEVRON USA INC	GI 0026	X	59	43	4	7
CHEVRON USA INC	GI 0037	R	58	49	7	7
CHEVRON USA INC	GI 0037	Y	60	50	7	7
CHEVRON USA INC	GI 0037	Z	84	50	7	7
CHEVRON USA INC	GI 0085	I	67	192	29	11
CHEVRON USA INC	GI 0086	AA	86	219	36	11
CHEVRON USA INC	MC 0063	B	83	480	13	9
CHEVRON USA INC	SP 0049	A	80	300	12	8
CHEVRON USA INC	SP 0049	C	81	250	10	8
CHEVRON USA INC	SP 0057	B	80	194	7	8
CHEVRON USA INC	SP 0058	D	81	180	7	8
CHEVRON USA INC	SP 0077	A	77	216	7	8
CHEVRON USA INC	SP 0077	C	80	240	7	8
CHEVRON USA INC	SS 0069	A	84	28	6	6
CHEVRON USA INC	SS 0069	B	84	28	6	6
CHEVRON USA INC	SS 0108	B	61	26	23	6
CHEVRON USA INC	SS 0108	CA	61	26	23	6
CHEVRON USA INC	SS 0108	CB	60	20	23	6
CHEVRON USA INC	SS 0108	CP	61	26	23	6
CHEVRON USA INC	SS 0108	CP	86	20	23	6
CHEVRON USA INC	SS 0108	D	89	26	23	6
CHEVRON USA INC	SS 0150	A	55	54	33	7
CHEVRON USA INC	SS 0154	C	56	60	36	7
CHEVRON USA INC	SS 0154	D	57	57	36	7
CHEVRON USA INC	SS 0154	E	56	54	36	7
CHEVRON USA INC	SS 0154	I	65	62	36	7
CHEVRON USA INC	SS 0154	J	66	62	36	7
CHEVRON USA INC	SS 0168	B	73	58	27	7

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
CHEVRON USA INC	SS 0168	B-PROD	73	58	27	7
CHEVRON USA INC	SS 0168	D	81	60	22	7
CHEVRON USA INC	SS 0169	A	61	54	28	7
CHEVRON USA INC	SS 0169	C	83	56	29	7
CHEVRON USA INC	SS 0170	A	79	58	33	7
CHEVRON USA INC	SS 0181	B	81	65	30	7
CHEVRON USA INC	SS 0181	B-PROD	83	65	30	7
CHEVRON USA INC	SS 0181	C	83	45	35	7
CHEVRON USA INC	SS 0181	D	87	65	30	7
CHEVRON USA INC	SS 0182	C	76	65	32	7
CHEVRON USA INC	SS 0182	C-PROD	77	65	32	7
CHEVRON USA INC	SS 0182	E	81	65	32	7
CHEVRON USA INC	SS 0183	F	83	66	33	7
CHEVRON USA INC	SS 0198	F	65	101	44	7
CHEVRON USA INC	SS 0198	H-DRILL	75	100	44	7
CHEVRON USA INC	SS 0198	H-PROD	76	100	44	7
CHEVRON USA INC	SS 0198	I	76	100	44	7
CHEVRON USA INC	SS 0198	J-DRILL	82	100	44	7
CHEVRON USA INC	SS 0198	J-PROD	85	100	44	7
CHEVRON USA INC	SS 0266	A	68	180	53	8
CHEVRON USA INC	SS 0266	B	71	178	53	8
CHEVRON USA INC	ST 0021	D	56	35	4	6
CHEVRON USA INC	ST 0021	E	73	14	4	6
CHEVRON USA INC	ST 0021	G	56	46	4	6
CHEVRON USA INC	ST 0021	H	56	35	4	6
CHEVRON USA INC	ST 0022	B	64	50	4	6
CHEVRON USA INC	ST 0022	C	66	48	4	6
CHEVRON USA INC	ST 0022	F	84	50	4	6
CHEVRON USA INC	ST 0023	S	57	44	5	7
CHEVRON USA INC	ST 0024	U	59	47	6	7
CHEVRON USA INC	ST 0027	D	66	50	7	7
CHEVRON USA INC	ST 0027	E	83	50	7	7
CHEVRON USA INC	ST 0027	I	66	48	7	7
CHEVRON USA INC	ST 0028	F	64	49	7	7
CHEVRON USA INC	ST 0028	PROD	64	49	7	7
CHEVRON USA INC	ST 0035	D	77	54	7	7
CHEVRON USA INC	ST 0035	E	78	52	7	7
CHEVRON USA INC	ST 0036	B	76	57	7	7
CHEVRON USA INC	ST 0036	G	82	47	7	7
CHEVRON USA INC	ST 0037	A	75	57	8	7
CHEVRON USA INC	ST 0037	C	77	52	8	7
CHEVRON USA INC	ST 0052	A	73	61	14	7
CHEVRON USA INC	ST 0052	C	85	61	14	8
CHEVRON USA INC	ST 0100	A	85	56	24	8
CHEVRON USA INC	ST 0128	A-AUX	68	103	27	11
CHEVRON USA INC	ST 0128	R	67	112	27	11
CHEVRON USA INC	ST 0128	X	82	100	26	11
CHEVRON USA INC	ST 0130	C	61	180	28	11
CHEVRON USA INC	ST 0130	D	62	160	28	11
CHEVRON USA INC	ST 0131	G	66	145	28	11
CHEVRON USA INC	ST 0131	J	69	176	28	11
CHEVRON USA INC	ST 0134	F	62	137	29	11
CHEVRON USA INC	ST 0134	N	67	130	29	11
CHEVRON USA INC	ST 0134	S	68	120	29	11
CHEVRON USA INC	ST 0134	T	68	130	29	11
CHEVRON USA INC	ST 0135	M	66	116	29	11
CHEVRON USA INC	ST 0135	Q	68	107	29	11
CHEVRON USA INC	ST 0135	V	75	122	29	11
CHEVRON USA INC	ST 0151	G	61	137	32	11
CHEVRON USA INC	ST 0151	I	63	128	32	11
CHEVRON USA INC	ST 0151	J	62	140	32	11
CHEVRON USA INC	ST 0151	K	64	137	32	11

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
CHEVRON USA INC	ST 0151	L	66	140	32	11
CHEVRON USA INC	ST 0151	O	67	137	32	11
CHEVRON USA INC	ST 0151	PROD-1	62	137	32	11
CHEVRON USA INC	ST 0151	PROD-2	62	137	32	11
CHEVRON USA INC	ST 0151	Y	88	137	32	11
CHEVRON USA INC	ST 0152	P	67	137	32	11
CHEVRON USA INC	ST 0176	A	63	140	35	11
CHEVRON USA INC	ST 0176	D	66	137	35	11
CHEVRON USA INC	ST 0177	E	67	141	35	11
CHEVRON USA INC	ST 0189	A	67	142	38	11
CHEVRON USA INC	ST 0190	A	77	142	38	11
CHEVRON USA INC	WD 0029	A	52	39	7	6
CHEVRON USA INC	WD 0029	B	52	38	7	6
CHEVRON USA INC	WD 0029	C	52	42	7	6
CHEVRON USA INC	WD 0029	D	52	39	7	6
CHEVRON USA INC	WD 0029	E	56	39	7	6
CHEVRON USA INC	WD 0029	F	55	39	7	6
CHEVRON USA INC	WD 0029	G	52	37	7	6
CHEVRON USA INC	WD 0029	H	56	39	7	6
CHEVRON USA INC	WD 0041	A	62	87	15	6
CHEVRON USA INC	WD 0041	B	64	83	15	6
CHEVRON USA INC	WD 0041	C	64	83	15	6
CHEVRON USA INC	WD 0117	C	65	214	34	10
CHEVRON USA INC	WD 0117	D	69	195	34	10
CHEVRON USA INC	WD 0117	E	69	208	34	10
CHEVRON USA INC	WD 0117	F	74	200	34	10
CHEVRON USA INC	WD 0117	G	79	211	34	10
CNG PRODUCING COMPANY	SS 0246	A	73	166	55	7
CNG PRODUCING COMPANY	SS 0246	B	74	170	55	7
CNG PRODUCING COMPANY	SS 0246	E	78	166	55	7
CNG PRODUCING COMPANY	SS 0247	C	74	180	57	7
CNG PRODUCING COMPANY	SS 0247	F	78	225	57	7
CNG PRODUCING COMPANY	SS 0248	D	75	180	59	7
CNG PRODUCING COMPANY	SS 0271	A	65	213	62	7
CNG PRODUCING COMPANY	SS 0271	B	78	214	62	7
CNG PRODUCING COMPANY	SS 0295	A	80	245	67	7
CNG PRODUCING COMPANY	ST 0075	E	88	65	18	7
CNG PRODUCING COMPANY	ST 0076	D	87	63	18	7
CNG PRODUCING COMPANY	ST 0076	F	90	63	18	7
CNG PRODUCING COMPANY	ST 0077	A	84	63	18	8
CNG PRODUCING COMPANY	ST 0077	B	84	63	18	8
CNG PRODUCING COMPANY	ST 0077	C	84	63	18	8
CONOCO INC	EI 0208	E	65	94	48	6
CONOCO INC	EI 0208	H	83	100	48	6
CONOCO INC	EI 0208	J	88	94	48	6
CONOCO INC	GC 0052	A	89	604	93	6
CONOCO INC	GC 0052	CPP	89	604	93	6
CONOCO INC	GI 0032	CC	91	92	16	8
CONOCO INC	GI 0032	J	65	106	18	8
CONOCO INC	GI 0040	B	56	83	14	8
CONOCO INC	GI 0040	F	60	86	14	8
CONOCO INC	GI 0040	G	68	85	14	8
CONOCO INC	GI 0040	I	70	86	14	8
CONOCO INC	GI 0041	A	64	91	17	8
CONOCO INC	GI 0041	B	66	91	17	8
CONOCO INC	GI 0041	D	68	90	17	8
CONOCO INC	GI 0041	E	68	85	17	8
CONOCO INC	GI 0041	H	88	91	20	8
CONOCO INC	GI 0042	C	67	100	19	8
CONOCO INC	GI 0042	F	75	100	19	8
CONOCO INC	GI 0043	AA-COMP	68	110	21	8
CONOCO INC	GI 0043	AA-PUMP	68	110	21	8

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
CONOCO INC	GI 0043	AA-QRTS	68	110	21	8
CONOCO INC	GI 0043	AA-RISER	68	110	21	8
CONOCO INC	GI 0043	AA-SEP	68	110	21	8
CONOCO INC	GI 0043	Q	67	140	21	8
CONOCO INC	GI 0047	A	55	89	18	8
CONOCO INC	GI 0047	AQ	72	89	18	8
CONOCO INC	GI 0047	C	57	88	18	8
CONOCO INC	GI 0047	H	70	90	18	8
CONOCO INC	GI 0047	L	83	100	18	8
CONOCO INC	GI 0048	D	59	86	15	8
CONOCO INC	GI 0048	E	58	91	15	8
CONOCO INC	GI 0048	J	73	24	15	8
CONOCO INC	GI 0048	K	76	90	15	8
CONOCO INC	SP 0075	A	86	356	7	8
CONOCO INC	SS 0145	B	66	43	23	7
CONOCO INC	SS 0145	D	85	45	23	7
CONOCO INC	SS 0158	C	80	50	26	7
CONOCO INC	SS 0198	G	67	106	44	7
CONOCO INC	SS 0198	K	85	98	44	7
CONOCO INC	SS 0232	A	80	110	40	8
CONOCO INC	ST 0146	A	79	97	30	10
CONOCO INC	WD 0040	A	66	83	15	6
CONOCO INC	WD 0040	B	69	90	15	6
CONOCO INC	WD 0044	D	57	49	12	6
CONOCO INC	WD 0045	A	55	49	12	6
CONOCO INC	WD 0045	B	55	49	12	6
CONOCO INC	WD 0045	C	56	50	12	6
CONOCO INC	WD 0045	CQ	62	50	12	6
CONOCO INC	WD 0045	E	57	50	12	6
CONOCO INC	WD 0045	F	59	53	12	6
CONOCO INC	WD 0045	G	59	49	12	6
CONOCO INC	WD 0045	H	70	60	12	6
CONOCO INC	WD 0058	C	80	50	13	6
CONOCO INC	WD 0058	D	82	52	13	6
CONOCO INC	WD 0068	U	70	111	19	8
CONOCO INC	WD 0069	C	62	121	23	8
CONOCO INC	WD 0069	F	65	135	23	8
CONOCO INC	WD 0069	K	66	134	23	8
CONOCO INC	WD 0070	D	63	131	23	8
CONOCO INC	WD 0070	H	65	141	23	8
CONOCO INC	WD 0070	I	65	135	23	8
CONOCO INC	WD 0070	L	66	135	23	8
CONOCO INC	WD 0070	N	67	138	23	8
CONOCO INC	WD 0070	Z	80	135	24	8
CONOCO INC	WD 0071	E	62	137	23	8
CONOCO INC	WD 0071	M	66	136	23	8
CONOCO INC	WD 0071	O	67	142	23	8
CONOCO INC	WD 0071	Y	78	149	23	8
CONOCO INC	WD 0094	G	64	153	27	9
CONOCO INC	WD 0094	V	70	156	27	9
CONOCO INC	WD 0095	S	68	150	27	9
CONOCO INC	WD 0095	T	68	150	27	9
CONOCO INC	WD 0095	X	73	150	27	9
CONOCO INC	WD 0096	R	67	148	27	9
CSX OIL & GAS CORPORATION	SS 0296	A	78	272	69	7
ELF AQUITAINE OPERATING INC	EI 0198	A	67	99	48	6
ELF AQUITAINE OPERATING INC	EI 0260	B	71	166	62	7
ELF AQUITAINE OPERATING INC	EI 0275	A	64	172	64	7
ELF AQUITAINE OPERATING INC	EI 0324	A	90	225	92	7
ELF AQUITAINE OPERATING INC	EI 0342	A	73	285	83	6
ELF AQUITAINE OPERATING INC	EI 0342	C	86	286	82	6
ELF AQUITAINE OPERATING INC	EI 0343	B	78	285	79	7

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
ELF AQUITAINE OPERATING INC	EW 0947	A	90	477	68	8
ELF AQUITAINE OPERATING INC	ST 0226	A	91	180	40	12
ELF EXPLORATION INC	EI 0184	A	90	80	26	6
ELF EXPLORATION INC	ST 0038	A	91	50	10	7
ELF EXPLORATION INC	WD 0138	A	78	306	42	11
ENERGY DEVELOPMENT CORPORATION	GI 0079	A	90	203	30	11
EXXON CORPORATION	EI 0251	A	78	160	54	7
EXXON CORPORATION	EI 0295	A	72	215	72	6
EXXON CORPORATION	EI 0295	A-PROD	72	215	72	6
EXXON CORPORATION	GI 0016	BB	81	34	11	7
EXXON CORPORATION	GI 0016	L-compressor	68	55	11	7
EXXON CORPORATION	GI 0016	L-quarters	67	55	11	7
EXXON CORPORATION	GI 0016	S	62	45	11	7
EXXON CORPORATION	GI 0021	W	66	65	11	7
EXXON CORPORATION	GI 0021	Z	67	62	11	7
EXXON CORPORATION	GI 0022	L-PROD	56	55	8	7
EXXON CORPORATION	GI 0022	P	57	55	8	7
EXXON CORPORATION	GI 0022	Q	61	55	8	7
EXXON CORPORATION	GI 0022	R	61	55	8	7
EXXON CORPORATION	GI 0022	U	63	60	8	7
EXXON CORPORATION	GI 0023	J	55	53	6	7
EXXON CORPORATION	GI 0023	K	56	50	6	7
EXXON CORPORATION	GI 0023	T	62	48	6	7
EXXON CORPORATION	MC 0268	A	78	343	44	11
EXXON CORPORATION	MC 0280	A	83	1000	22	9
EXXON CORPORATION	MC 0397	A	91	468	46	12
EXXON CORPORATION	SP 0093	A	77	446	18	10
EXXON CORPORATION	SP 0093	B	84	436	20	10
EXXON CORPORATION	SS 0111	A	90	46	14	6
EXXON CORPORATION	SS 0322	A	84	310	75	9
EXXON CORPORATION	ST 0054	1038#2	68	61	13	8
EXXON CORPORATION	ST 0054	G	82	67	13	8
EXXON CORPORATION	ST 0054	G-quarters	87	67	13	8
EXXON CORPORATION	ST 0055	E	58	67	14	8
EXXON CORPORATION	ST 0055	F	76	69	14	8
EXXON CORPORATION	ST 0067	B	55	66	16	8
EXXON CORPORATION	ST 0164	C	78	99	33	10
EXXON CORPORATION	ST 0165	A	69	93	34	9
EXXON CORPORATION	ST 0165	A-PROD	70	93	34	9
EXXON CORPORATION	ST 0165	E	80	91	34	9
EXXON CORPORATION	ST 0170	F	81	97	42	9
EXXON CORPORATION	ST 0171	B	77	100	36	9
EXXON CORPORATION	ST 0171	D	79	106	36	9
EXXON CORPORATION	ST 0172	A-quarters	71	93	36	9
EXXON CORPORATION	WD 0030	E	54	52	7	6
EXXON CORPORATION	WD 0030	E-compressor	63	52	7	6
EXXON CORPORATION	WD 0030	E-quarters	62	52	7	6
EXXON CORPORATION	WD 0030	G	55	39	7	6
EXXON CORPORATION	WD 0030	H	55	49	7	6
EXXON CORPORATION	WD 0030	J	55	45	7	6
EXXON CORPORATION	WD 0030	L	56	53	7	6
EXXON CORPORATION	WD 0030	N	56	55	7	6
EXXON CORPORATION	WD 0030	P	57	43	7	6
EXXON CORPORATION	WD 0030	S-compressor	68	54	7	6
EXXON CORPORATION	WD 0030	T	62	50	7	6
EXXON CORPORATION	WD 0030	Z	83	35	7	6
EXXON CORPORATION	WD 0031	E	54	52	8	6
EXXON CORPORATION	WD 0031	F	55	47	8	6
EXXON CORPORATION	WD 0031	H	55	49	8	6
EXXON CORPORATION	WD 0031	L	56	53	8	6
EXXON CORPORATION	WD 0031	N	57	55	8	6
EXXON CORPORATION	WD 0032	Q	57	56	9	6

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
EXXON CORPORATION	WD 0032	S	62	54	9	6
EXXON CORPORATION	WD 0032	U	62	57	9	6
EXXON CORPORATION	WD 0042	X	68	74	14	6
EXXON CORPORATION	WD 0042	Y	67	82	14	6
EXXON CORPORATION	WD 0073	A	64	168	22	8
EXXON CORPORATION	WD 0073	A-Tower	62	168	22	8
EXXON CORPORATION	WD 0073	C	64	172	22	8
EXXON CORPORATION	WD 0073	D	65	168	22	8
EXXON CORPORATION	WD 0073	D-Quarters	67	170	22	8
EXXON CORPORATION	WD 0074	B	64	180	22	8
EXXON CORPORATION	WD 0074	F	65	170	22	8
EXXON CORPORATION	WD 0091	G	86	186	23	9
EXXON CORPORATION	WD 0093	E	65	160	27	9
EXXON CORPORATION	WD 0099	A	63	194	31	9
EXXON CORPORATION	WD 0099	B	81	200	31	9
EXXON CORPORATION	WD 0100	C	89	197	21	9
FMP OPERATING COMPANY A LIMIT	SP 0045	A	86	189	20	7
FMP OPERATING COMPANY A LIMIT	WD 0105	E	87	237	40	9
FOREST OIL CORPORATION	EI 0190	A	87	70	57	6
FOREST OIL CORPORATION	EI 0273	A	70	191	68	6
FOREST OIL CORPORATION	EI 0284	B	84	193	69	6
FOREST OIL CORPORATION	EI 0286	I	85	215	75	6
FOREST OIL CORPORATION	EI 0325	A	89	257	68	7
FOREST OIL CORPORATION	EI 0366	A	89	337	76	6
FOREST OIL CORPORATION	SS 0277	A	89	218	51	7
FREEPORT-MCMORAN OIL & GAS	GI 0083	A	80	153	33	11
FREEPORT-MCMORAN OIL & GAS	MC 0365	A	91	600	21	10
FREEPORT-MCMORAN OIL & GAS	SP 0083	A	90	472	17	9
FREEPORT-MCMORAN RESOURCE PART	GI 0017	#1 PROD	69	55	6	6
FREEPORT-MCMORAN RESOURCE PART	GI 0017	#2 PROD	69	55	6	6
GAS TRANSPORATION CORP	SP 0037	C	61	105	7	7
GAS TRANSPORATION CORP	SP 0037	E	83	105	7	7
HALL-HOUSTON OIL COMPANY	EI 0278	F	83	172	65	7
HALL-HOUSTON OIL COMPANY	EI 0281	A	90	197	59	6
HALL-HOUSTON OIL COMPANY	GI 0082	A	89	203	30	11
HALL-HOUSTON OIL COMPANY	ST 0221	A	89	156	46	11
HARBERT ENERGY CORPORATION	SS 0190	A	90	76	32	8
HUGHES EASTERN PETROLEUM INC	SP 0037	C	64	160	7	7
KERR-MCGEE CORPORATION	SS 0214	B	65	110	37	8
KERR-MCGEE CORPORATION	SS 0214	C	66	100	37	8
KERR-MCGEE CORPORATION	SS 0214	D	67	100	37	8
KERR-MCGEE CORPORATION	SS 0214	E	67	105	37	8
KERR-MCGEE CORPORATION	SS 0214	F	67	105	37	8
KERR-MCGEE CORPORATION	SS 0214	G	65	110	37	8
KERR-MCGEE CORPORATION	SS 0214	H	70	115	37	8
KERR-MCGEE CORPORATION	SS 0214	J	71	110	37	8
KERR-MCGEE CORPORATION	SS 0214	K	80	107	37	8
KERR-MCGEE CORPORATION	SS 0214	L	83	115	37	8
KERR-MCGEE CORPORATION	SS 0218	B	84	113	39	8
KERR-MCGEE CORPORATION	SS 0229	A	69	130	45	8
KERR-MCGEE CORPORATION	SS 0229	B	71	125	45	8
KERR-MCGEE CORPORATION	SS 0229	C	82	130	45	8
KERR-MCGEE CORPORATION	SS 0230	A	62	120	42	8
KERR-MCGEE CORPORATION	SS 0230	B	72	126	42	8
KERR-MCGEE CORPORATION	SS 0233	B	81	128	39	8
KERR-MCGEE CORPORATION	SS 0238	A	81	129	43	8
KERR-MCGEE CORPORATION	SS 0238	B	82	129	43	8
KERR-MCGEE CORPORATION	SS 0239	A	68	133	45	8
KERR-MCGEE CORPORATION	ST 0034	B	84	50	9	7
KERR-MCGEE CORPORATION	ST 0050	A	81	58	10	7
KERR-MCGEE CORPORATION	ST 0050	B	81	58	10	7
KERR-MCGEE CORPORATION	ST 0050	C	80	58	10	7

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
KERR-MCGEE CORPORATION	ST 0197	A	90	120	40	10
KIRBY EXPLORATION COMPANY OF	WD 0134	D	65	280	39	10
MARATHON OIL COMPANY	EI 0349	A	74	320	82	6
MARATHON OIL COMPANY	EI 0349	B	79	320	82	6
MARATHON OIL COMPANY	SP 0089	A	78	450	14	10
MARATHON OIL COMPANY	WD 0079	A	67	110	17	8
MARATHON OIL COMPANY	WD 0079	B	70	132	17	8
MARATHON OIL COMPANY	WD 0079	C	69	150	17	8
MARATHON OIL COMPANY	WD 0079	E	84	115	17	8
MARATHON OIL COMPANY	WD 0079	F	88	110	17	8
MARATHON OIL COMPANY	WD 0080	D	71	102	13	9
MARATHON OIL COMPANY	WD 0086	A	82	150	15	9
MARATHON OIL COMPANY	WD 0086	B	90	192	6	9
MARATHON OIL COMPANY	WD 0143	A	90	369	18	10
MESA OPERATING LIMITED PART	WD 0061	A	78	110	15	7
MESA PETROLEUM CO	PL 0013	A	77	35	10	7
MESA PETROLEUM CO	PL 0013	B	78	35	10	7
MESA PETROLEUM CO	PL 0013	S	78	35	10	7
MOBIL OIL EXPLORATION & PRODUC	EI 0240	A	83	139	42	6
MOBIL OIL EXPLORATION & PRODUC	GC 0018	A	86	750	96	7
MOBIL OIL EXPLORATION & PRODUC	GI 0020	1	83	40	10	7
MOBIL OIL EXPLORATION & PRODUC	GI 0020	A	82	60	10	7
MOBIL OIL EXPLORATION & PRODUC	GI 0090	A	85	225	40	12
MOBIL OIL EXPLORATION & PRODUC	GI 0093	C	75	205	43	12
MOBIL OIL EXPLORATION & PRODUC	GI 0094	B	74	210	40	12
MOBIL OIL EXPLORATION & PRODUC	GI 0095	A	73	210	43	12
MOBIL OIL EXPLORATION & PRODUC	PL 0010	A	76	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0010	B	79	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0010	C	79	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0010	D	80	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0010	E	82	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0010	LQ	79	35	5	7
MOBIL OIL EXPLORATION & PRODUC	PL 0011	F	57	37	8	7
MOBIL OIL EXPLORATION & PRODUC	SS 0072	A	48	29	6	6
MOBIL OIL EXPLORATION & PRODUC	SS 0072	I	78	20	6	6
MOBIL OIL EXPLORATION & PRODUC	SS 0072	J	78	45	6	6
MOBIL OIL EXPLORATION & PRODUC	SS 0087	B	50	31	8	6
MOBIL OIL EXPLORATION & PRODUC	SS 0182	A	82	70	32	7
MOBIL OIL EXPLORATION & PRODUC	SS 0182	B	84	70	32	7
MOBIL OIL EXPLORATION & PRODUC	SS 0182	C	86	70	32	7
NERCO OIL & GAS INC	EI 0217	B	89	110	38	6
NERCO OIL & GAS INC	EI 0371	B	87	415	95	6
NERCO OIL & GAS INC	SS 0202	A	88	112	54	7
NEWFIELD EXPLORATION COMPANY	SS 0157	A	91	52	30	7
NEWFIELD EXPLORATION COMPANY	SS 0197	A	91	100	40	7
ODECO OIL & GAS COMPANY	EI 0265	A	91	174	56	6
ODECO OIL & GAS COMPANY	PL 0012	A	79	32	9	7
ODECO OIL & GAS COMPANY	PL 0012	E	91	29	7	7
ODECO OIL & GAS COMPANY	PL 0019	B	87	32	12	7
ODECO OIL & GAS COMPANY	PL 0019	OBM	61	32	12	7
ODECO OIL & GAS COMPANY	SS 0113	N	67	48	13	7
ODECO OIL & GAS COMPANY	SS 0114	B-AUX	70	32	13	7
ODECO OIL & GAS COMPANY	SS 0114	H	67	52	13	7
ODECO OIL & GAS COMPANY	SS 0114	J	79	32	13	7
ODECO OIL & GAS COMPANY	SS 0114	L	89	32	13	7
ODECO OIL & GAS COMPANY	SS 0118	E	58	54	16	7
ODECO OIL & GAS COMPANY	SS 0120	K	79	50	17	6
ODECO OIL & GAS COMPANY	SS 0167	#8 PLATFORM	90	50	27	7
ODECO OIL & GAS COMPANY	SS 0223	B	75	144	52	7
ODECO OIL & GAS COMPANY	SS 0224	A	71	147	54	7
ODECO OIL & GAS COMPANY	SS 0224	D	79	156	54	7
ODECO OIL & GAS COMPANY	SS 0224	E	78	147	54	7

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
ODECO OIL & GAS COMPANY	ST 0063	A	57	98	18	9
ODECO OIL & GAS COMPANY	ST 0086	B	77	92	21	9
ODECO OIL & GAS COMPANY	ST 0086	C	77	92	21	9
ODECO OIL & GAS COMPANY	ST 0086	D	86	92	21	9
OXY USA INC	EI 0327	A	76	262	78	6
PENNZOIL COMPANY	EI 0330	A	71	244	82	6
PENNZOIL COMPANY	EI 0330	B	71	248	82	6
PENNZOIL COMPANY	EI 0330	C	72	247	82	6
PENNZOIL COMPANY	EI 0330	D	86	248	82	6
PENNZOIL COMPANY	EI 0330	S	72	247	82	6
PENNZOIL EXPLORATION AND PROD	EI 0256	A	77	137	53	7
PENNZOIL EXPLORATION AND PROD	EI 0261	A	76	166	62	7
PENNZOIL EXPLORATION AND PROD	EI 0316	A	84	240	79	6
PENNZOIL EXPLORATION AND PROD	SP 0078	A	78	227	6	8
PHILLIPS PETROLEUM COMPANY	SS 0130	E	83	47	30	6
PHILLIPS PETROLEUM COMPANY	SS 0133	A	82	38	20	6
PHILLIPS PETROLEUM COMPANY	SS 0133	CA	84	38	20	6
PHILLIPS PETROLEUM COMPANY	SS 0149	A	56	55	33	7
PHILLIPS PETROLEUM COMPANY	SS 0149	C	73	55	33	7
PHILLIPS PETROLEUM COMPANY	SS 0149	D	80	42	33	7
PHILLIPS PETROLEUM COMPANY	SS 0149	G	91	55	33	7
PLACID OIL COMPANY	SS 0204	A	68	100	43	7
PLACID OIL COMPANY	SS 0207	A	67	100	34	8
PLACID OIL COMPANY	SS 0207	B	68	100	34	8
PLACID OIL COMPANY	SS 0207	D	73	100	34	8
PLACID OIL COMPANY	SS 0216	C	70	110	38	8
PSI ENERGY RESOURCES INC	EI 0191	A	84	97	58	6
PSI INC	SS 0058	A	91	15	14	6
PSI INC	SS 0058	A AUX	89	15	14	6
PSI INC	ST 0162	A	66	120	33	10
PSI INC	ST 0162	B	90	112	33	10
SAMEDAN OIL CORPORATION	EI 0208	F	65	94	48	6
SAMEDAN OIL CORPORATION	EI 0208	I	84	91	48	6
SAMEDAN OIL CORPORATION	EI 0248	B	84	164	60	6
SAMEDAN OIL CORPORATION	EI 0248	C	84	164	60	6
SAMEDAN OIL CORPORATION	SS 0080	A	88	24	20	6
SAMEDAN OIL CORPORATION	ST 0147	A	87	100	33	10
SAMEDAN OIL CORPORATION	ST 0163	A	76	105	33	10
SAMEDAN OIL CORPORATION	ST 0172	B	80	99	37	10
SAMEDAN OIL CORPORATION	ST 0172	C	82	99	37	10
SAMEDAN OIL CORPORATION	ST 0186	B	79	172	40	11
SAMEDAN OIL CORPORATION	ST 0192	D	82	115	35	10
SAMEDAN OIL CORPORATION	ST 0195	A	81	113	38	10
SAMEDAN OIL CORPORATION	ST 0196	A	71	105	37	10
SAMEDAN OIL CORPORATION	ST 0196	B	89	105	34	10
SANDEFER OFFSHORE OPERATING	EI 0277	A	90	165	50	7
SHELL OFFSHORE INC	EI 0188	A	58	65	38	6
SHELL OFFSHORE INC	EI 0188	C	67	65	30	6
SHELL OFFSHORE INC	EI 0259	A	64	170	58	7
SHELL OFFSHORE INC	EI 0259	B	66	170	58	7
SHELL OFFSHORE INC	EI 0259	C	68	160	58	7
SHELL OFFSHORE INC	GC 0019	A	88	750	90	7
SHELL OFFSHORE INC	GC 0065	A(BULLWINKLE)	88	1353	90	8
SHELL OFFSHORE INC	GI 0033	A	86	86	14	8
SHELL OFFSHORE INC	GI 0075	JA	79	146	28	11
SHELL OFFSHORE INC	GI 0076	A	73	150	31	11
SHELL OFFSHORE INC	MC 0194	A	78	1023	15	9
SHELL OFFSHORE INC	MC 0311	A	78	425	46	11
SHELL OFFSHORE INC	SP 0028	T	66	36	4	6
SHELL OFFSHORE INC	SP 0028	TT	66	36	4	6
SHELL OFFSHORE INC	SP 0028	V	60	56	4	6
SHELL OFFSHORE INC	SP 0028	Z	60	75	4	6

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)

ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
SHELL OFFSHORE INC	SP 0070	C	74	264	9	7
SHELL OFFSHORE INC	SP 0070	D	77	290	9	7
SHELL OFFSHORE INC	SS 0189	A	83	65	31	8
SHELL OFFSHORE INC	SS 0274	A	64	210	64	7
SHELL OFFSHORE INC	SS 0274	C	89	210	64	7
SHELL OFFSHORE INC	SS 0293	B	70	236	64	7
SHELL OFFSHORE INC	ST 0026	A	65	55	8	7
SHELL OFFSHORE INC	ST 0026	C	71	60	8	7
SHELL OFFSHORE INC	ST 0026	D	71	60	8	7
SHELL OFFSHORE INC	ST 0026	E	74	62	8	7
SHELL OFFSHORE INC	ST 0026	F	79	60	8	7
SHELL OFFSHORE INC	ST 0295	A	85	282	60	10
SHELL OFFSHORE INC	ST 0300	A	81	337	60	10
SHELL OFFSHORE INC	ST 0301	B	82	379	60	10
SHELL OFFSHORE INC	WD 0032	A	62	63	9	6
SHELL OFFSHORE INC	WD 0032	B	63	60	9	6
SHELL OFFSHORE INC	WD 0032	C	64	59	9	6
SHELL OFFSHORE INC	WD 0032	E	71	63	9	6
SHELL OFFSHORE INC	WD 0103	A	65	223	27	9
SHELL OFFSHORE INC	WD 0103	B	65	228	27	9
SHELL OFFSHORE INC	WD 0104	C	65	228	27	9
SHELL OFFSHORE INC	WD 0104	D	67	260	27	9
SHELL OIL COMPANY	EI 0189	B	64	65	35	6
SONAT EXPLORATION COMPANY	SS 0222	A	69	120	49	7
SONAT EXPLORATION COMPANY	SS 0222	D	69	120	49	7
SONAT EXPLORATION COMPANY	SS 0225	B	71	146	54	7
SONAT EXPLORATION COMPANY	SS 0225	C	71	146	53	7
SONAT EXPLORATION COMPANY	SS 0225	E	84	146	54	7
STONE PETROLEUM CORPORATION	PL 0023	CA	62	61	15	7
TAYLOR ENERGY COMPANY	WD 0133	B	66	285	37	10
TAYLOR ENERGY COMPANY	WD 0133	E	67	292	37	10
TEAXACO AND EXPLORATION PROD	SP 0054	A	70	280	13	8
TEAXACO AND EXPLORATION PROD	WD 0076	A	83	180	18	8
TEXACO INC	EI 0339	B	73	260	83	6
TEXACO INC	EI 0339	C	74	268	83	6
TEXACO INC	EI 0365	A	78	333	84	6
TEXACO INC	EI 0367	A	75	349	80	7
TEXACO INC	WD 0109	A	80	184	12	9
TORCH OPERATING COMPANY	EI 0296	A	72	210	71	6
TORCH OPERATING COMPANY	EI 0296	B	72	214	71	6
TORCH OPERATING COMPANY	EI 0296	B-PROD	72	214	71	6
TORCH OPERATING COMPANY	EI 0306	A	72	225	75	6
TORCH OPERATING COMPANY	EI 0306	B	73	225	75	6
TORCH OPERATING COMPANY	SS 0204	B	69	107	43	7
TORCH OPERATING COMPANY	SS 0291	A	76	240	95	7
TORCH OPERATING COMPANY	WD 0152	A	70	373	47	11
UNION EXPLORATION PARTNERS LTD	EI 0297	A	85	200	74	7
UNION EXPLORATION PARTNERS LTD	SS 0215	C	62	110	37	8
UNION EXPLORATION PARTNERS LTD	SS 0215	I	70	112	37	8
UNION PACIFIC RESOURCES CO	SS 0165	A	85	60	25	7
UNION PACIFIC RESOURCES CO	SS 0191	A	89	89	34	8
UNION PACIFIC RESOURCES CO	ST 0198	A	90	130	44	10
UNION TEXAS PETROLEUM CORP	EI 0384	A	79	431	93	6
UNION TEXAS PETROLEUM CORP	ST 0148	A	76	110	29	10
UNION TEXAS PETROLEUM CORP	ST 0185	A	73	180	40	12
UNION TEXAS PETROLEUM CORP	ST 0185	B	77	175	40	12
UNOCAL EXPLORATION CORPORATION	EI 0212	A	85	86	66	7
UNOCAL EXPLORATION CORPORATION	EI 0276	B PROD	65	172	63	7
UNOCAL EXPLORATION CORPORATION	EI 0276	C	66	168	63	7
UNOCAL EXPLORATION CORPORATION	EI 0276	D	68	176	63	7
UNOCAL EXPLORATION CORPORATION	EI 0276	E	82	175	69	7
UNOCAL EXPLORATION CORPORATION	EI 0372	A	84	414	95	6

Table A-9: Platforms in Path of Andrew which Survived (Without Damage)						
ABBREVIATED OPERATOR NAME	AREA BLOCK	STRUCTURE NAME	YEAR INSTALLED	WATER DEPTH	DISTANCE FROM SHORE (MILES)	Hs (M)
UNOCAL EXPLORATION CORPORATION	SS 0208	E	63	103	34	8
UNOCAL EXPLORATION CORPORATION	SS 0208	F	64	97	34	8
UNOCAL EXPLORATION CORPORATION	SS 0208	H	68	98	34	8
UNOCAL EXPLORATION CORPORATION	SS 0208	PURE	64	97	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	A	72	95	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	B	61	100	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	BRP	71	100	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	D	63	95	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	G	67	100	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	J	71	94	34	8
UNOCAL EXPLORATION CORPORATION	SS 0209	K	73	107	34	8
UNOCAL EXPLORATION CORPORATION	SS 0253	A	62	165	54	8
UNOCAL EXPLORATION CORPORATION	SS 0253	C	69	175	54	8
UNOCAL EXPLORATION CORPORATION	SS 0253	D	70	170	54	8
UNOCAL EXPLORATION CORPORATION	SS 0253	E	81	165	54	8
UNOCAL EXPLORATION CORPORATION	SS 0268	C	91	201	52	7
UNOCAL EXPLORATION CORPORATION	SS 0269	A	65	170	59	7
UNOCAL EXPLORATION CORPORATION	SS 0269	B	72	205	59	7
UNOCAL EXPLORATION CORPORATION	ST 0053	A	80	62	13	8
W & T OFFSHORE INC	SS 0219	A	69	112	47	8
W & T OFFSHORE INC	SS 0219	C	72	113	47	8
WALTER OIL & GAS CORPORATION	SS 0160	A	89	50	25	7
WALTER OIL & GAS CORPORATION	WD 0062	A	89	120	26	7
WALTER OIL & GAS CORPORATION	WD 0063	A	80	125	15	7
WALTER OIL & GAS CORPORATION	WD 0063	B	90	140	15	7
ZILKHA ENERGY COMPANY	EI 0322	A	78	235	68	7
ZILKHA ENERGY COMPANY	EI 0322	A-PROD	78	235	68	7
ZILKHA ENERGY COMPANY	WD 0097	A	91	156	23	9

Appendix B
Calibration Results Details

This Appendix includes the following:

- Figure B-1: Path of hurricane Andrew with platforms used in calibration
- Table B-1 to Table B-3: Summary of platform characteristics and seastate data for the platforms

- Details for each platform: The following details are included for each platform

  Platform orientation and analysis directions.
  Configuration of vertical framings
  Base shear coefficients
  Approximate estimate of hindcast base shear in each direction with storm hours
  Selected results from pushover analysis

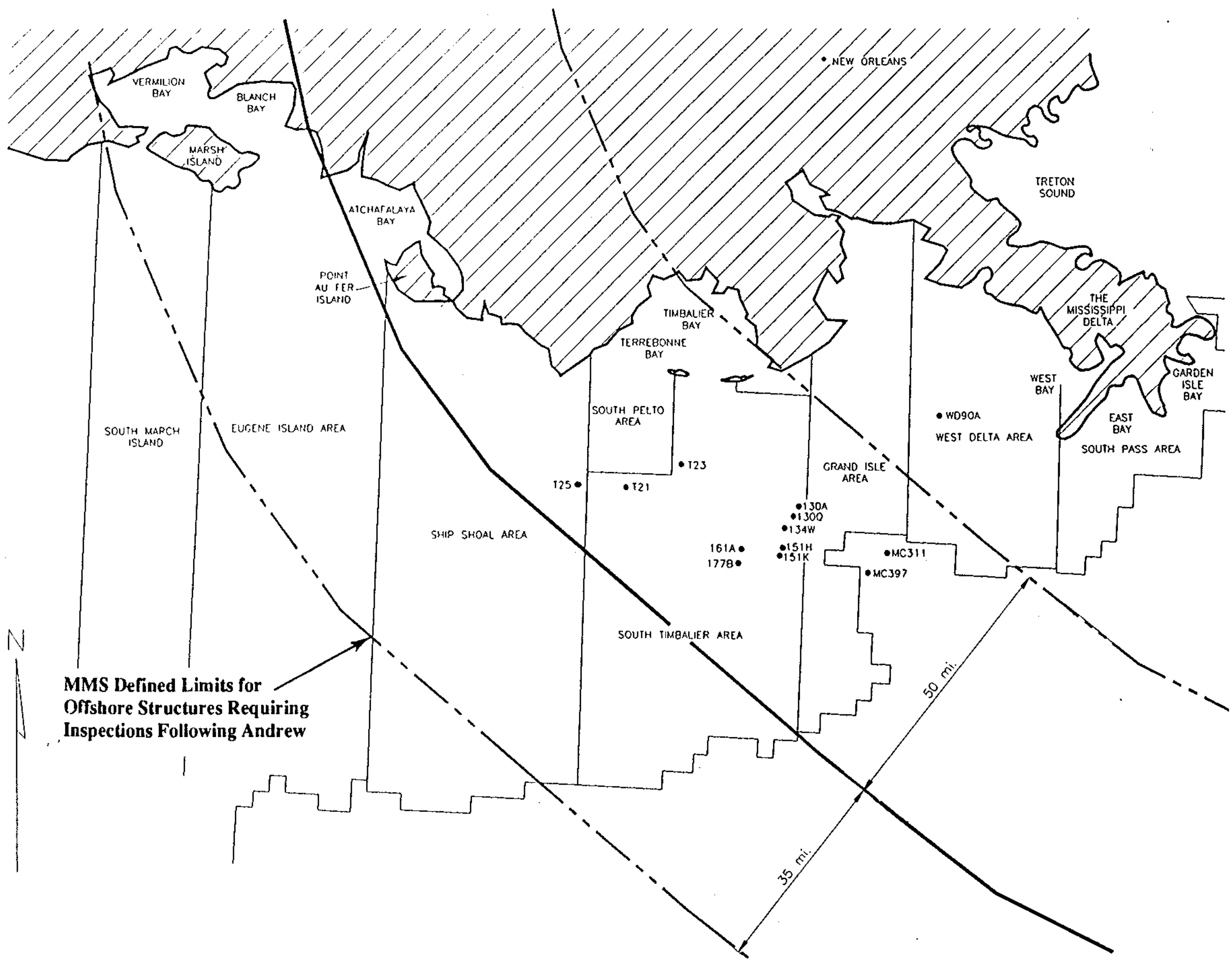


Figure B-1 Path of Hurricane Andrew with Platforms Used in Calibration

Table B-1: Platforms Selected for Calibration - Andrew JIP

Platform Operator	Platform Name	Platform Location- Block	Water Depth ft.	Year of Installation	Number of legs	Performance in Andrew	Pushover Analysis Performed by
Survival Platform Cases:							
Chevron	ST151K	South Timbalier-151	137	1963	8 leg	Survived	PMB
Chevron	ST130Q	South Timbalier-130	170	1964	4 leg	Survived	PMB
Chevron	ST134W	South Timbalier-134	137	1981	4 leg	Damaged *	PMB
Amoco	WD90A	West Delta-90	184	1964	8 leg	Survived	PMB
Exxon	MC311	Mississippi Canyon-311	343	1978	8 main legs, 8-skirt legs	Survived	Exxon
Exxon	MC397	Mississippi Canyon-397	468	1991	4 main legs, 4-skirt legs	Survived	Exxon
Damage Platform Cases:							
Trunkline	T23ST52	South Timbalier-52	63	1969	4 leg	Minor Damage	PMB
Trunkline	T25SS139	Ship Shoal-139	62	1969	4 leg	Heavily Damaged	PMB
Amoco	ST161A	South Timbalier-161	118	1964	8 leg	Minor Damage	Amoco
Failure Platform Cases:							
Chevron	ST177B	South Timbalier-177	142	1965	8 leg	Heavily Damaged/ Salvaged **	PMB
Chevron	ST151H	South Timbalier-151	137	1964	8 leg	Collapsed	PMB
Chevron	ST130A	South Timbalier-130	140	1958	8 leg	Collapsed	Chevron
Trunkline	T21ST72	South Timbalier-72	61	1969	4 leg	Collapsed	PMB

Notes:
* ST134W Platform considered as a survival case, as analysis did not reflect that hurricane alone could have caused the observed damage.
** ST177B platform was heavily damaged in Andrew. This platform has now been removed and salvaged.

Table B-2 Summary of Physical Characteristics of Platforms

Platform Name	Water Depth ft.	Year of Installation	Physical Characteristics						
			Number of legs & batter	Leg-Pile annulus grouted/ ungrouted	Brace type in broadside frames	Brace type in endon frames	Sub-cellar deck elevation ft.	Cellar deck elevation ft.	Main deck elevation ft.
Survival Platform Cases:									
ST151K	137	1963	8-double	grouted	diagonal	K-braces	-	36.500	46.875
ST130Q	170	1964	4-double	grouted	K-braces	K-braces	-	39.000	50.500
ST134W	137	1981	4-double at 2legs	ungrouted	diagonal	diagonal	-	43.000	60.000
WD90A	184	1964	8-double	grouted	diag. & K	K-braces	26.750	34.833	50.104
MC311	343	1978	8-double	ungrouted	diagonal	X-braces	-	57.000	80.950
MC397	468	1991	4-double	ungrouted	X-braces	X-braces	53.500	65.000	93.000
Damage Platform Cases:									
T23ST52	63	1969	4-double	grouted	K-braces	K-braces	-	52.594	68.521
T25SS139	62	1969	4-double	grouted	K-braces	K-braces	-	50.594	67.240
ST161A	118	1964	8-double	grouted	diagonal	K-braces	-	34.750	49.945
Failure Platform Cases:									
ST177B	142	1965	8-double	grouted	diagonal	K-braces	-	39.000	50.500
ST151H	137	1964	8-single	grouted	diagonal	K-braces	-	39.500	48.625
ST130A	140	1958	8-single	grouted	diagonal	K-braces	-	55.000	67.000
T21ST72	61	1969	4-double	grouted	K-braces	K-braces	-	54.594	71.240

Table B-3: Summary of Hindcast Maximum Seastate During Andrew at Platform Locations

Platform Name	Water Depth ft.	Year of Installation	Sea State Data: Hindcast maximum wave height H ft.	Hindcast maximum current U ft/sec	Hindcast maximum crest height Hc ft.	Extreme wave dir. from true North degree	1st and 2nd Direction Considered in Calibration	Remarks
Survival Platform Cases:								
ST151K	137	1963	60.854	3.443	40.198	286.40	Broadside & Diagonal	Wave in cellar deck
ST130Q	170	1964	62.269	3.000	41.341	289.10	Diagonal	Wave in cellar deck
ST134W	137	1981	60.854	3.443	40.198	286.40	Endon & Diagonal	-
WD90A	184	1964	50.531	2.560	33.457	303.36	Diagonal & Endon	Wave in sub-cellar deck
MC311	343	1978	62.000	2.384	40.000	288.30	Broadside & Diagonal	-
MC397	468	1991	65.911	1.330	40.727	273.40	Broadside & Diagonal	-
Damage Platform Cases:								
T23ST52	63	1969	50.300	4.130	31.382	286.37	Diagonal	-
T25SS139	62	1969	50.630	4.250	30.920	257.70	Orthogonal	-
ST161A	118	1964	58.495	3.560	40.228	290.40	Broadside	Wave in cellar deck
Failure Platform Cases:								
ST177B	142	1965	60.167	3.460	41.800	267.60	Diagonal-1	Wave in cellar deck
ST151H	137	1964	60.854	3.443	40.198	286.40	Diagonal	Wave in cellar deck
ST130A	140	1958	60.950	3.410	40.290	286.62	Diagonal	-
T21ST72	61	1969	49.740	4.260	31.072	257.31	Orthogonal	-

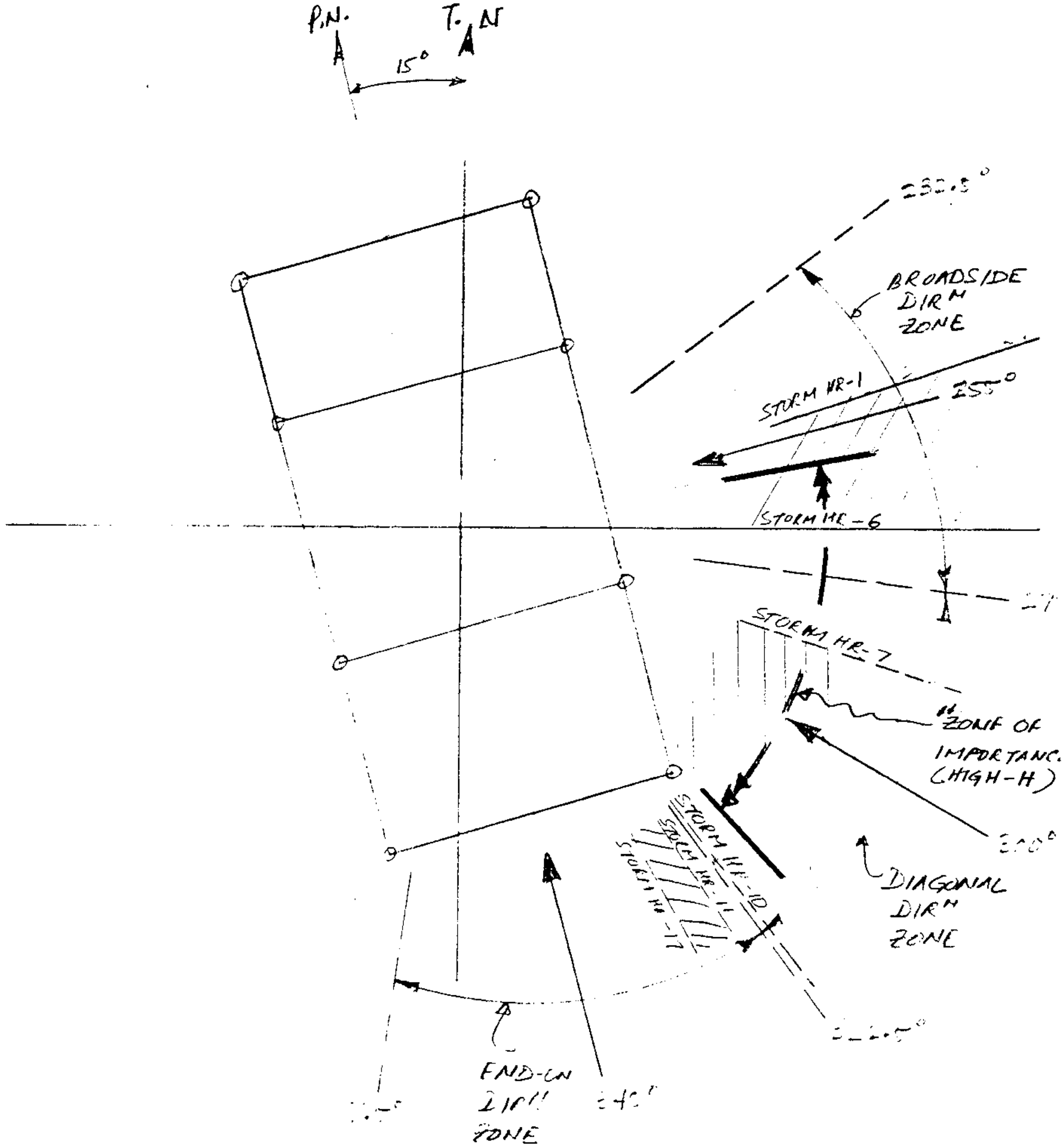
Platform ST151K

PMB ENGINEERING

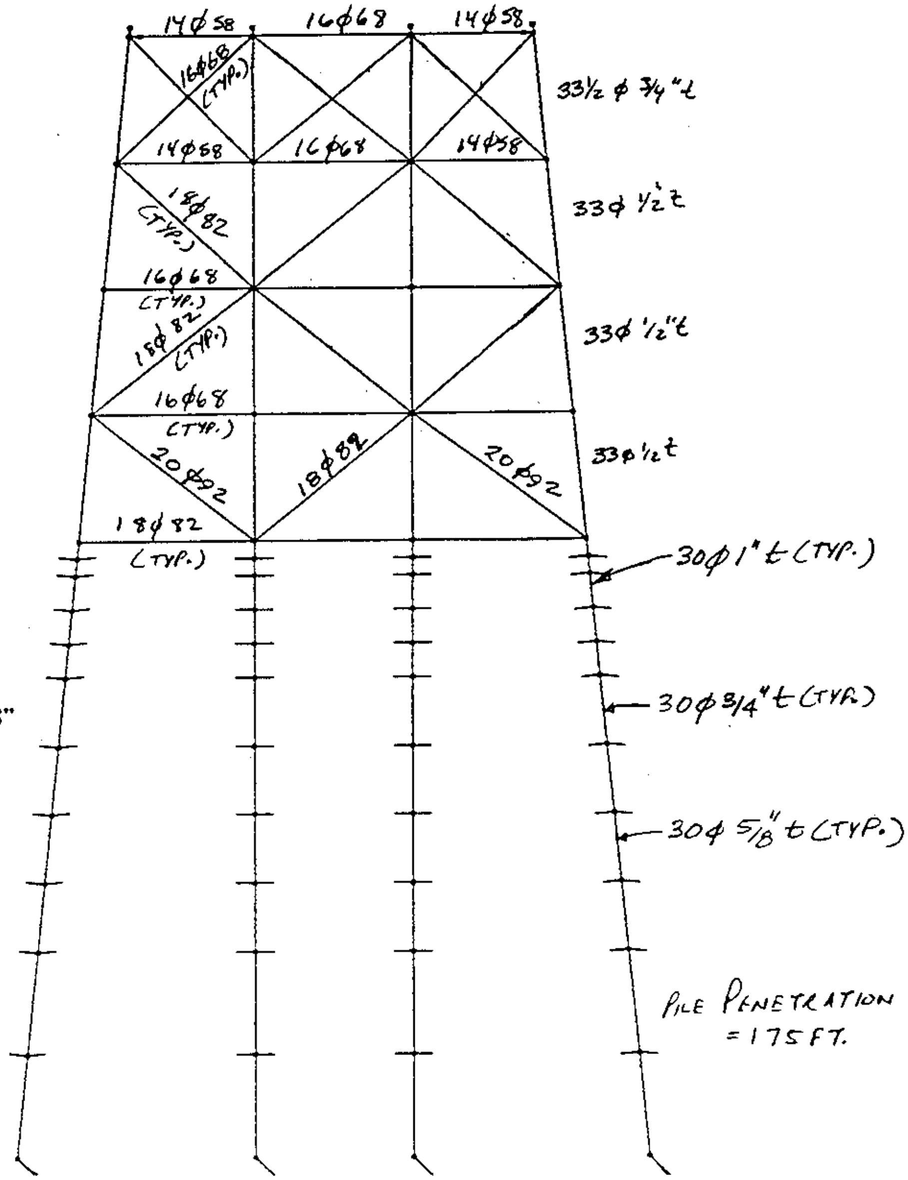
By: RKA Date: 05/25/93 Checked by: ____ Sheet No. ____

Project: ANDREW JIP Job No. 295

Subject: PLATFORM ST 151K



PLATFORM ORIENTATION, STORM APPROACH DIRECTIONS, AND ANALYZED DIRECTION

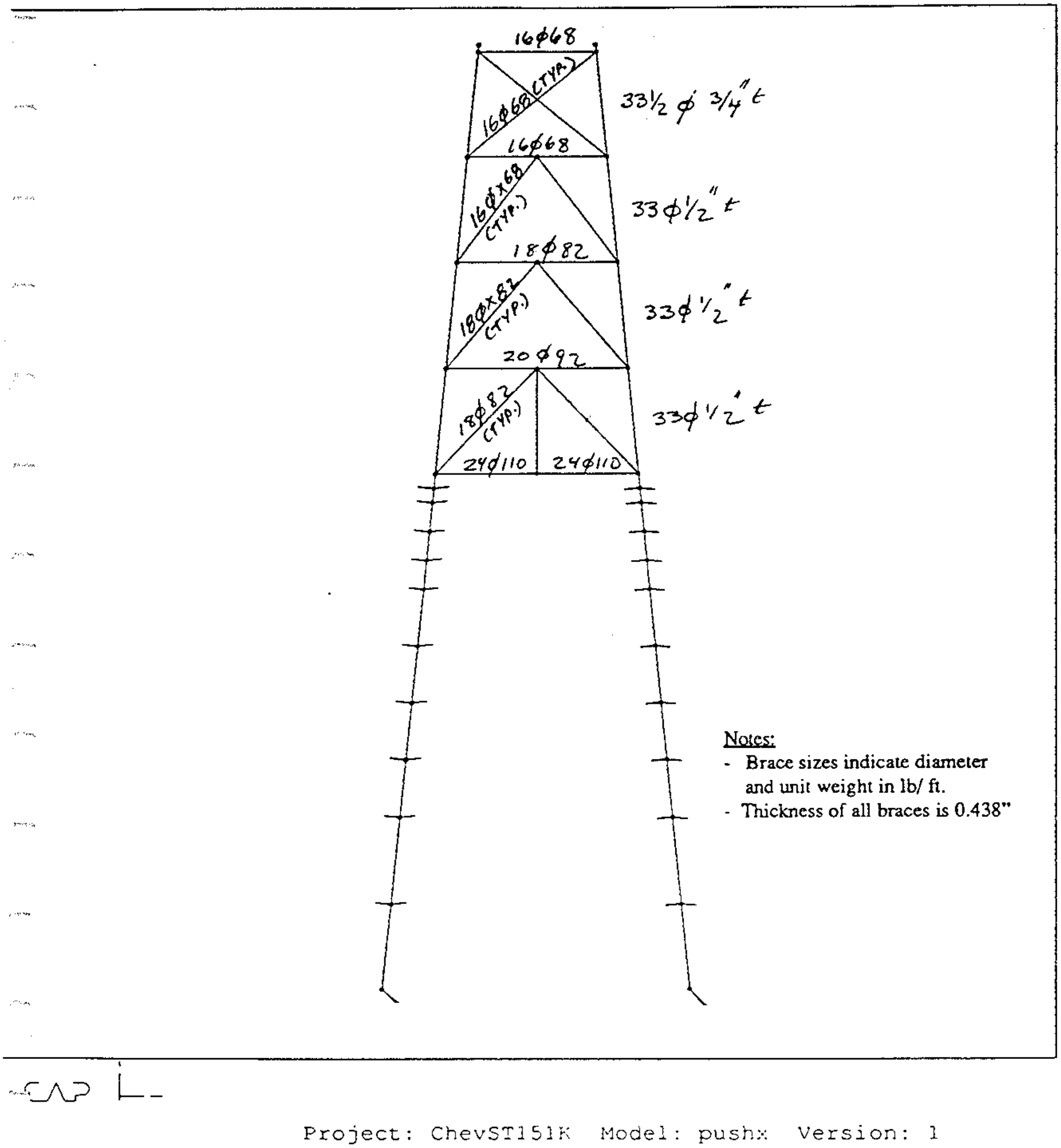


Notes:
- Brace sizes indicate diameter and unit weight in lb/ ft.
- Thickness of all braces is 0.438"

CAP

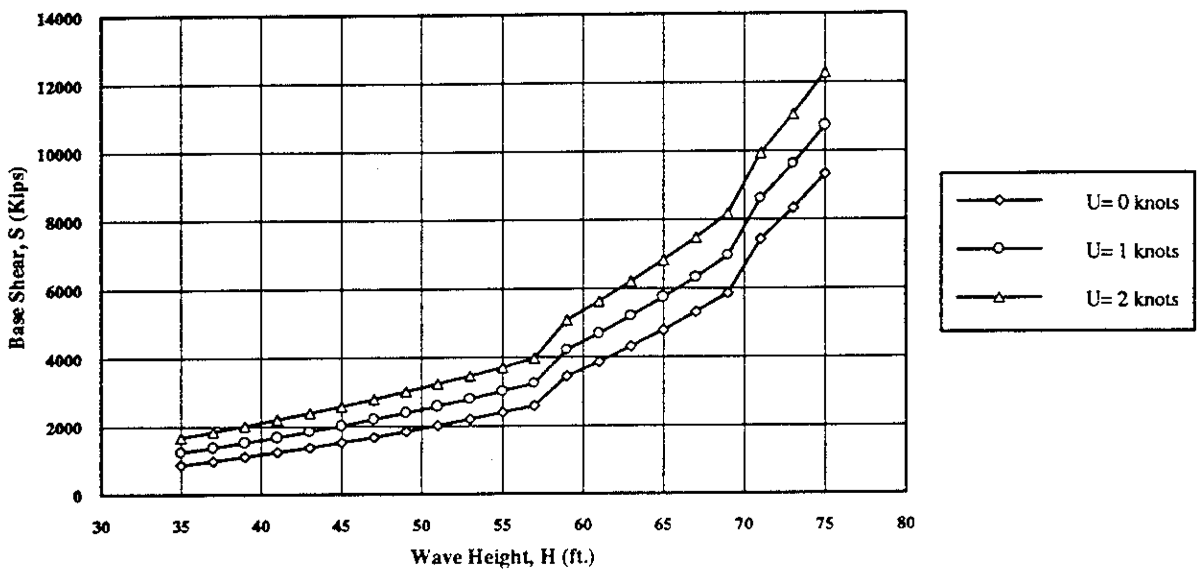
Project: ChevST151K Model: pushx Version: 1

Structural Configuration: Rows A & B - ST151K



Structural Configuration: Rows 1 to 4 - ST151K

Wave Height vs Base Shear: ST151K, Broadside Direction



C1, C2, C3 for Platform ST151K

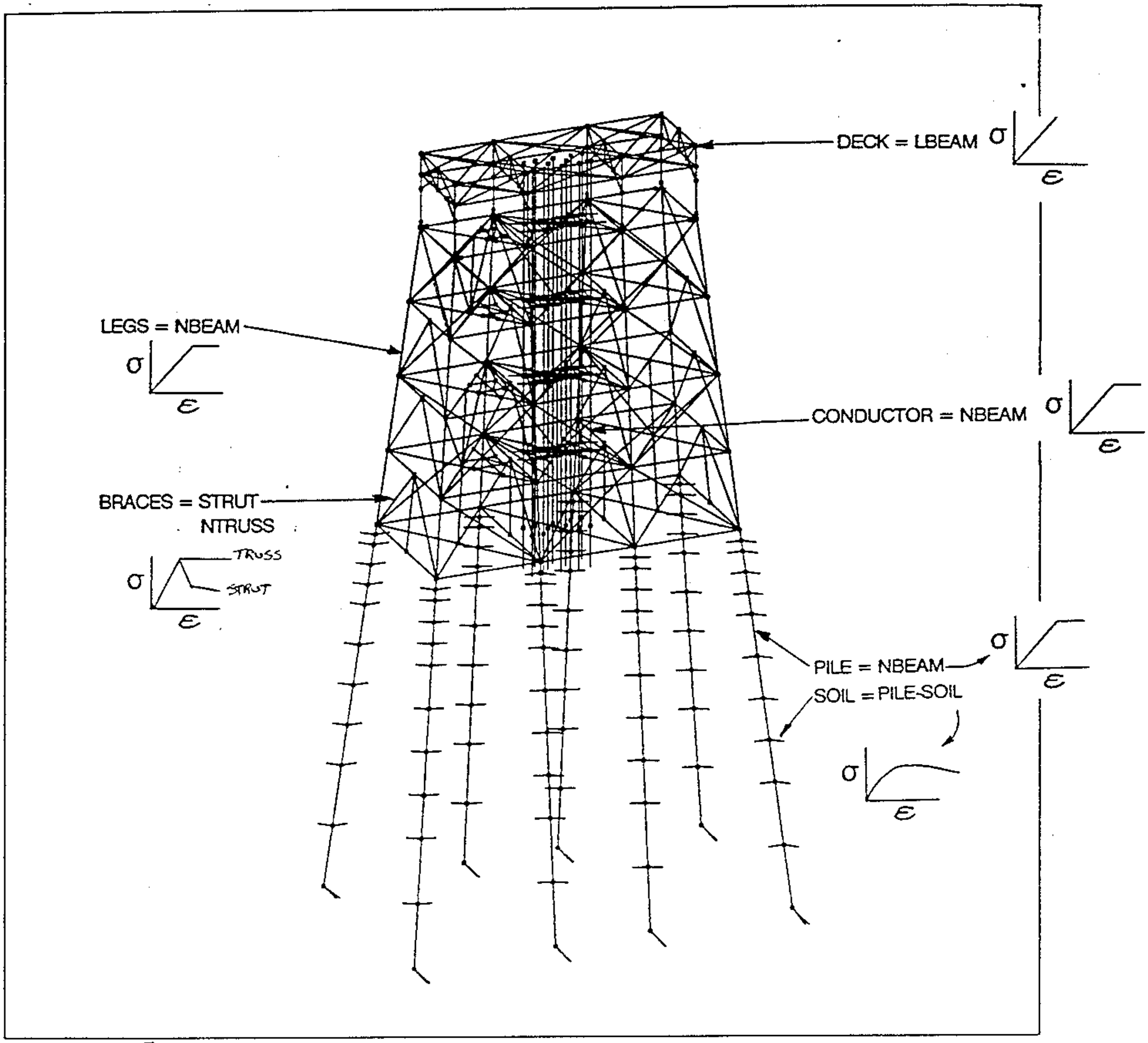
Direction	Wave Height Range ft.	C1	C2	C3
Broadside	35 to 57	0.301	5.99	2.24
	58 to 69	3.56E-03	3.60	3.38
	70 to 75	1.54E-04	2.59	4.15
Diagonal	35 to 57	0.169	6.21	2.36
	58 to 69	2.87E-03	4.10	3.38
	70 to 75	2.28E-02	4.57	2.90
End-On	35 to 57	0.154	5.33	2.36
	58 to 69	6.23E-03	3.86	3.17
	70 to 75	1.80E-02	4.05	2.92

PLATFORM ST151K

This Platform Survived Against Hurricane Andrew Loads

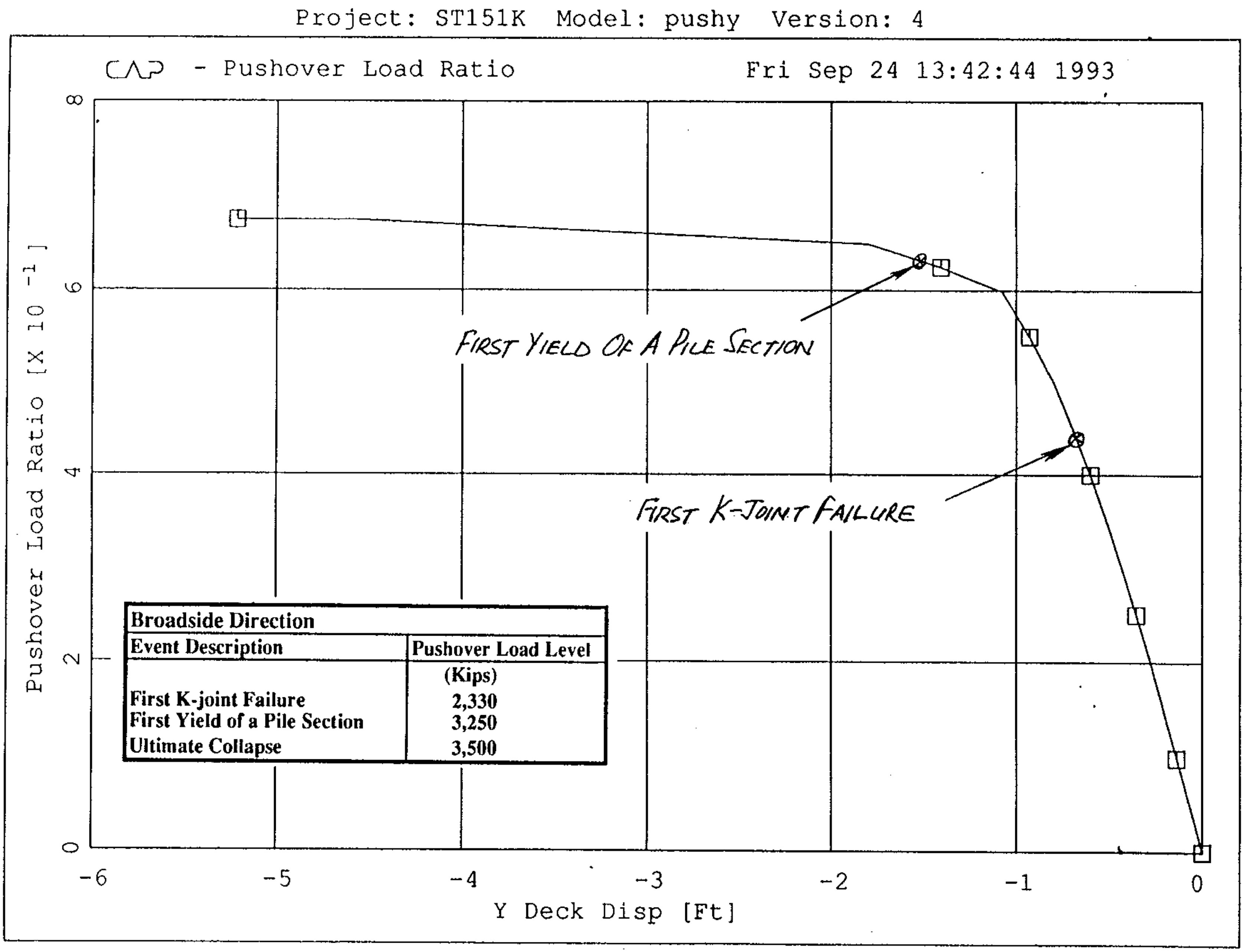
Water Depth = 137 ft.

Storm Hour	Wave Direction (Degree)	Hindcast Data: Hs (ft.)	H max = Hs * 1.683 (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	U (knots)	Base Shear Coeficients: C1	C2	C3	H+C2*U	Expected Maximum Base Shear (Kips)	Remarks	R median (Kips)
Broad-Side Direction			(232.5 deg. to 277.5 deg.)										3500.00
1	251.9	18.11	30.476	9.94	7.35	0.60	0.301	5.99	2.24	34.049	813.74		
2	253.3	20.79	34.984	10.88	8.05	0.75	0.301	5.99	2.24	39.467	1132.73		
3	252.9	23.18	39.011	11.25	8.32	0.95	0.301	5.99	2.24	44.708	1497.75		
4	255.0	25.96	43.682	11.91	8.81	1.21	0.301	5.99	2.24	50.933	2005.62		
5	258.3	29.64	49.882	12.72	9.42	1.53	0.301	5.99	2.24	59.055	2793.76		
6	268.2	33.89	57.029	13.67	10.12	1.87	0.00356	3.60	3.38	63.748	4472.54	Wave in Deck	
Diagonal Direction			(277.5 deg. to 322.5 deg.)										3500.00
7	286.4	36.16	60.854	14.45	10.69	2.04	0.00287	4.10	3.38	69.230	4765.26	Wave in Deck	
8	305.8	33.45	56.288	13.85	10.25	1.77	0.169	6.21	2.36	67.259	3478.36	Wave in Deck	
9	314.2	31.06	52.277	12.95	9.58	1.34	0.169	6.21	2.36	60.623	2722.14		
10	317.7	28.97	48.762	12.25	9.06	0.92	0.169	6.21	2.36	54.472	2114.71		
11	321.6	27.50	46.278	11.76	8.70	0.52	0.169	6.21	2.36	49.522	1688.94		
End On Direction			(322.5 deg. to 367.5 deg.)										
12	324.9	25.79	43.410	11.40	8.44	0.31	0.154	5.33	2.36	45.040	1230.32		
13	327.4	24.01	40.412	10.92	8.08	0.12	0.154	5.33	2.36	41.028	987.19		
14	329.2	22.54	37.927	10.67	7.90	0.02	0.154	5.33	2.36	38.009	824.25		
15	330.8	21.20	35.681	10.52	7.78	0.00	0.154	5.33	2.36	35.681	710.02		
16	331.7	19.98	33.620	10.25	7.58	0.00	0.154	5.33	2.36	33.620	617.02		
17	331.7	18.66	31.409	9.87	7.30	0.00	0.154	5.33	2.36	31.409	525.51		



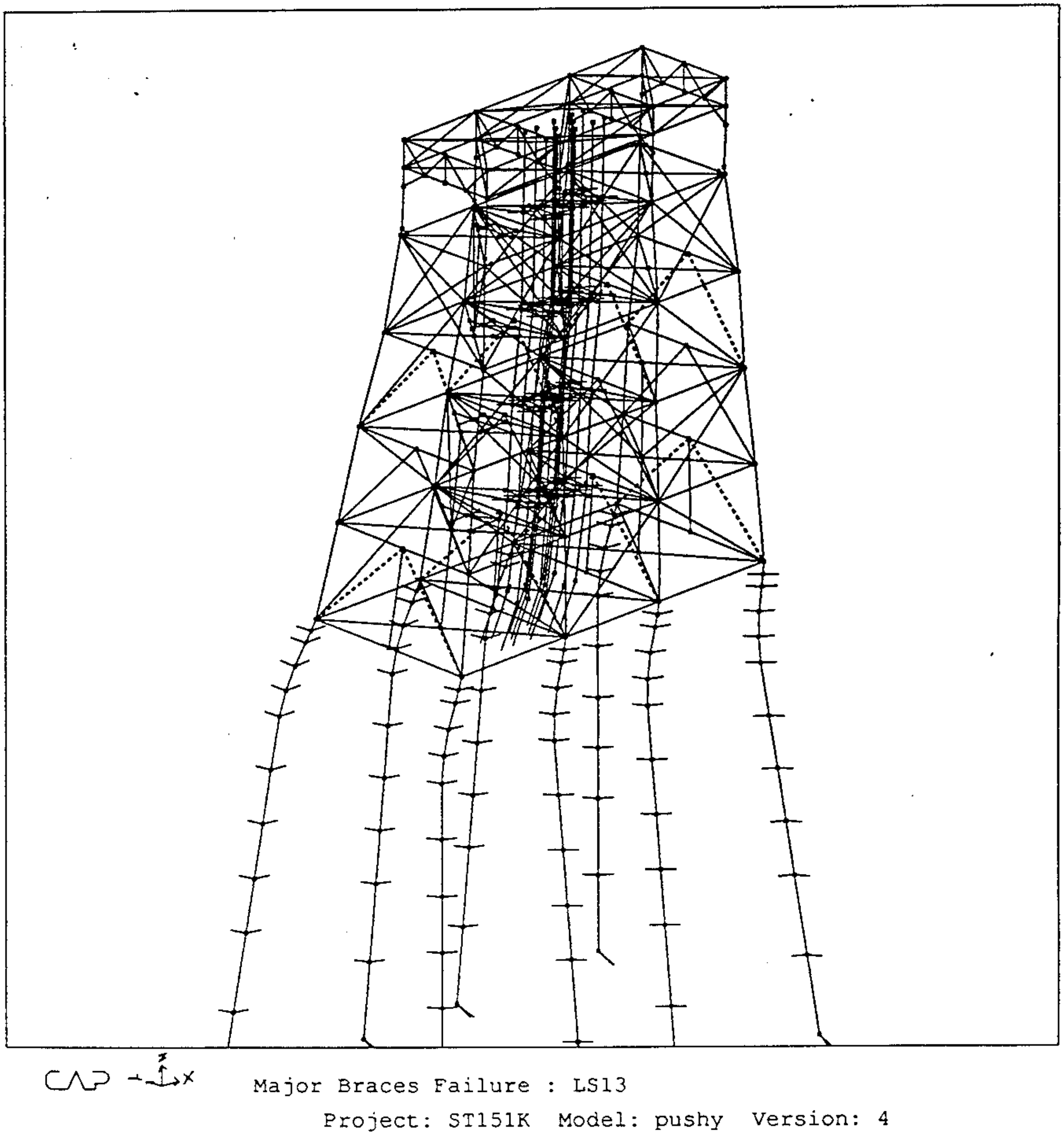
CAP z x Chevron ST151K Pushover Model - Simplified
Project: ChevST151K Model: pushy Version: 1

Nonlinear Analysis Computer Model - ST151K

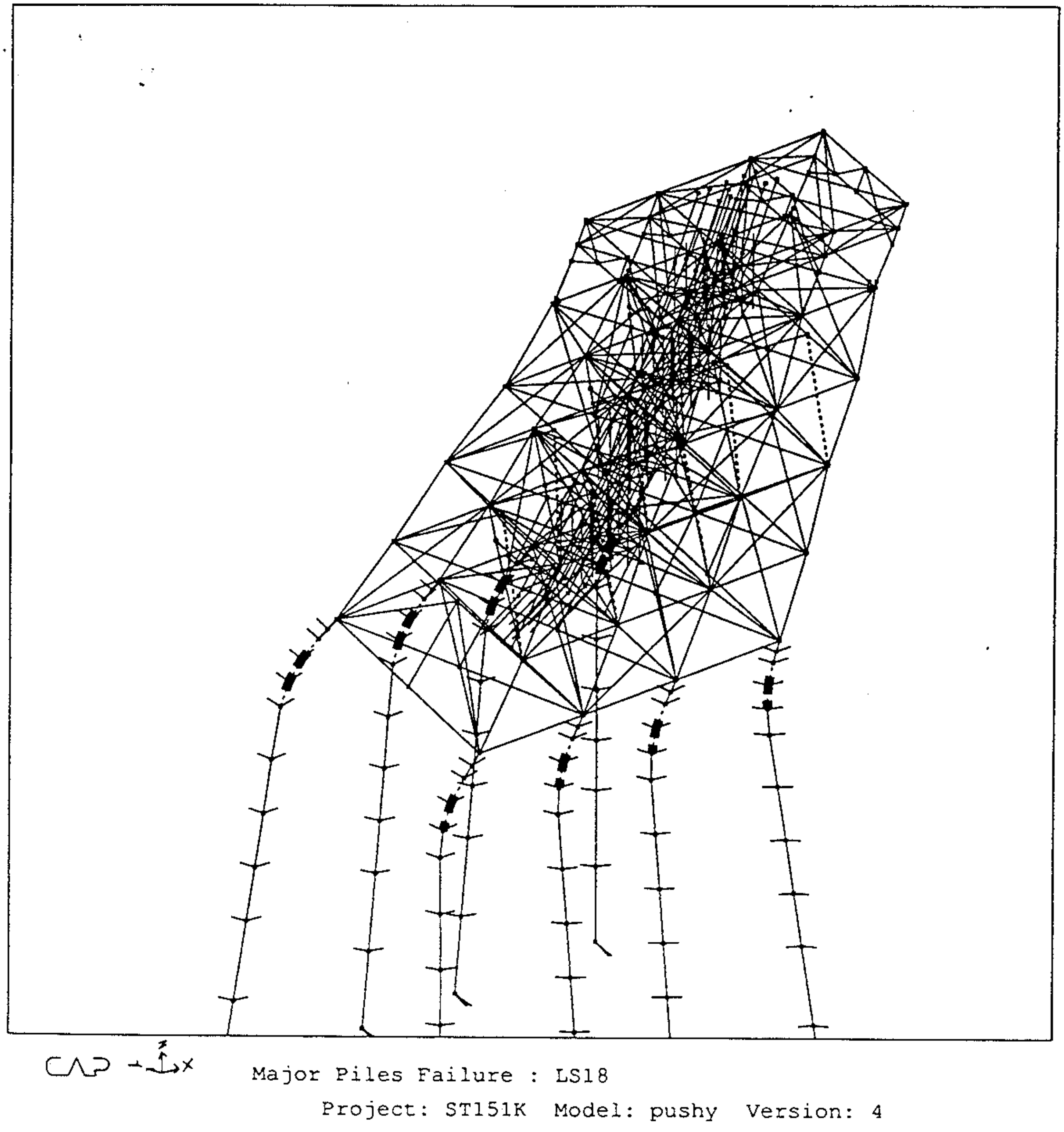


Broadside Direction	
Event Description	Pushover Load Level (Kips)
First K-joint Failure	2,330
First Yield of a Pile Section	3,250
Ultimate Collapse	3,500

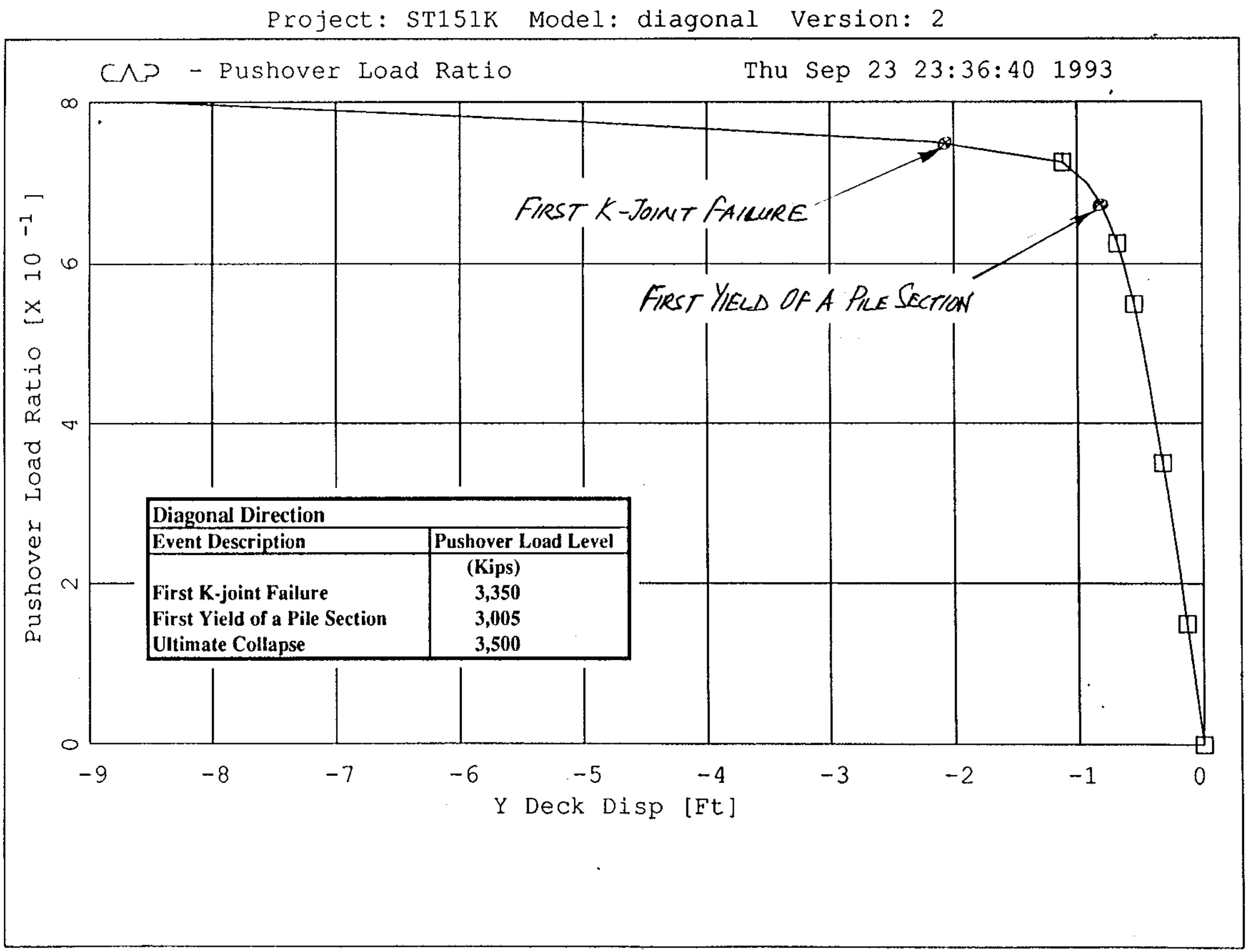
Pushover Analysis Results: Broadside Direction - ST151K



Pushover Analysis Joint/ Brace Failures: Broadside Direction - ST151K

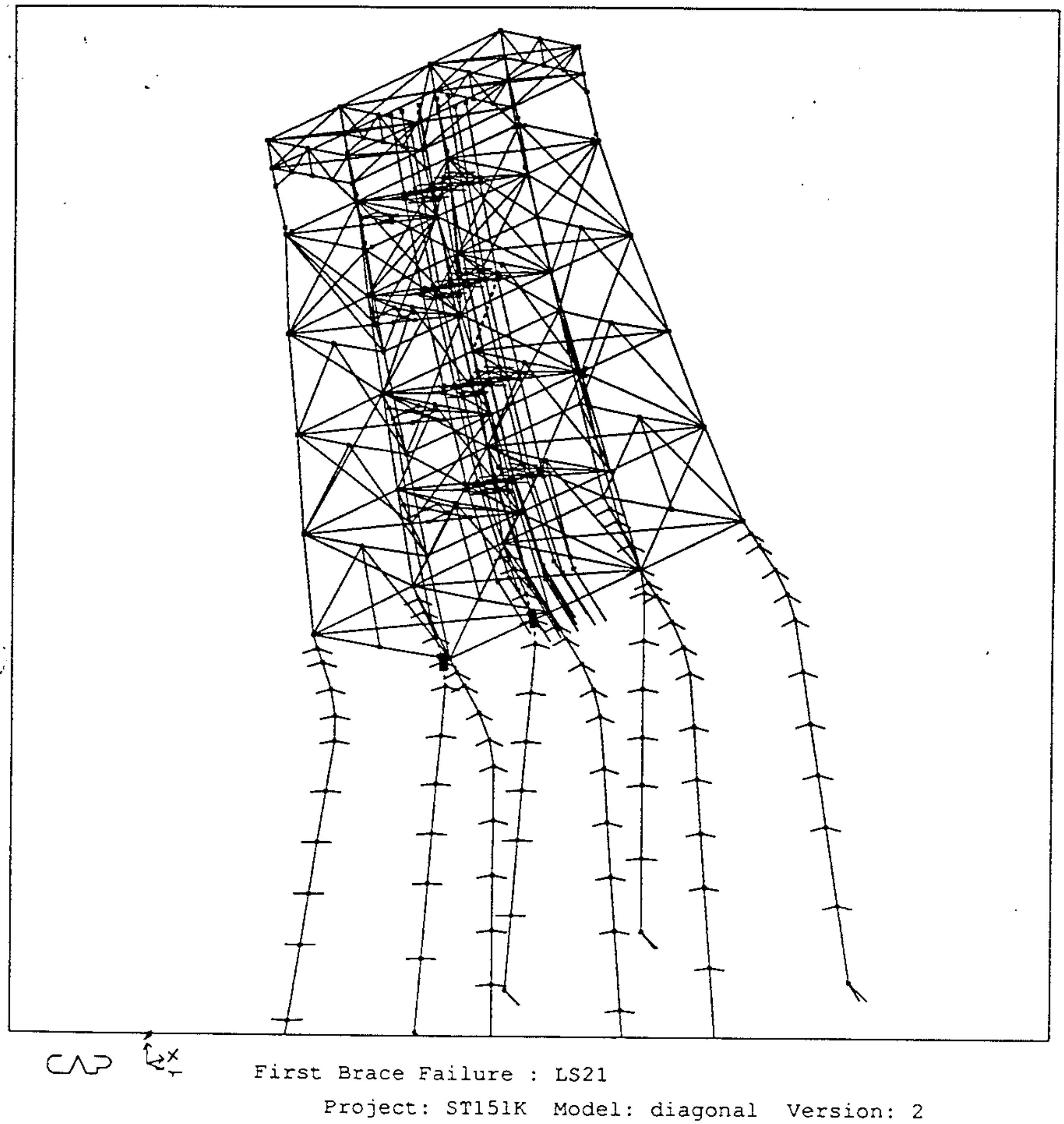


Pushover Analysis Ultimate Collapse Results: Broadside Direction - ST151K

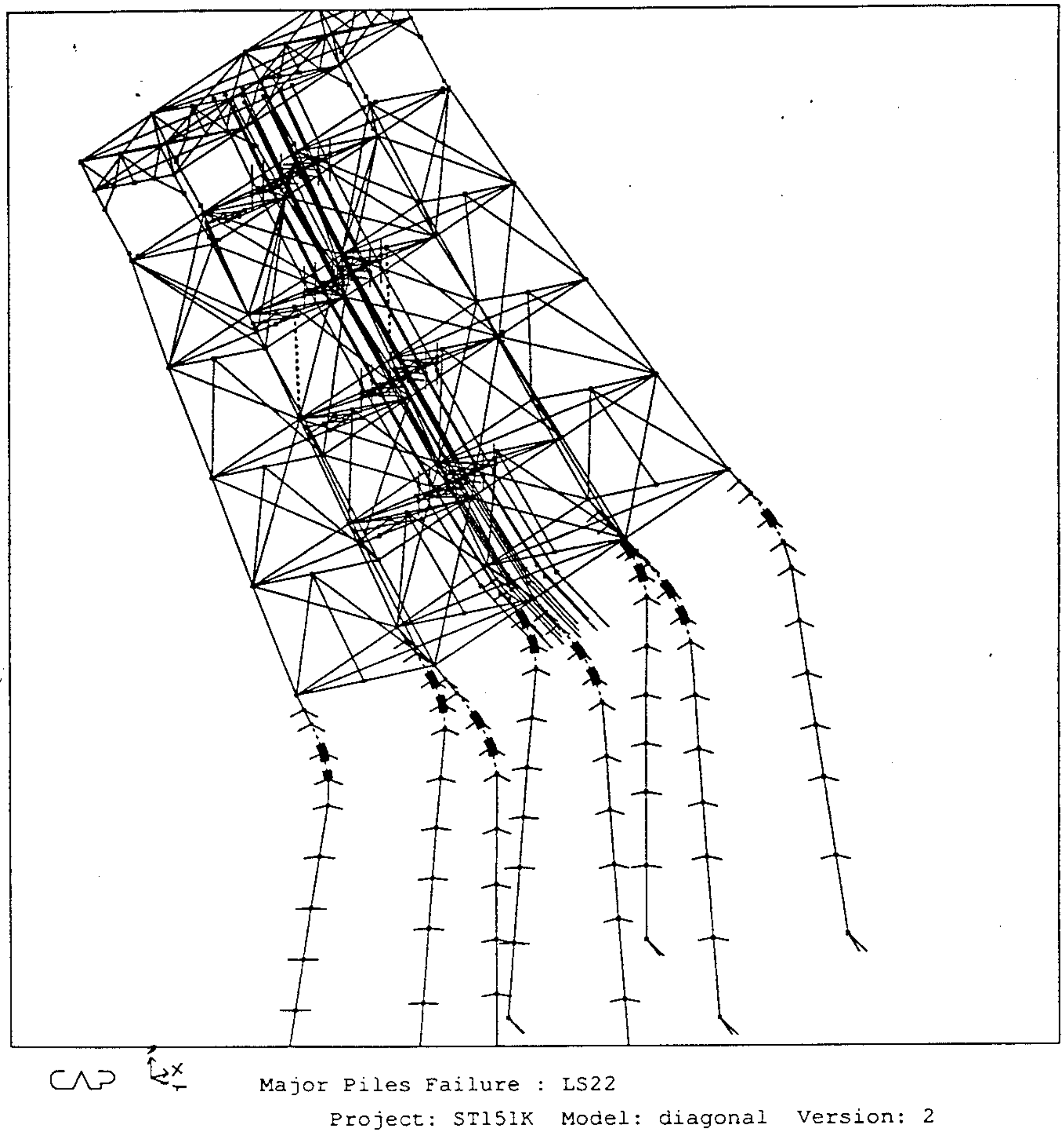


Diagonal Direction	
Event Description	Pushover Load Level (Kips)
First K-joint Failure	3,350
First Yield of a Pile Section	3,005
Ultimate Collapse	3,500

Pushover Analysis Results: Diagonal Direction - ST151K



Pushover Analysis First Member Failures: Diagonal Direction - ST151K



Pushover Analysis Ultimate Collapse Results: Diagonal Direction - ST151K

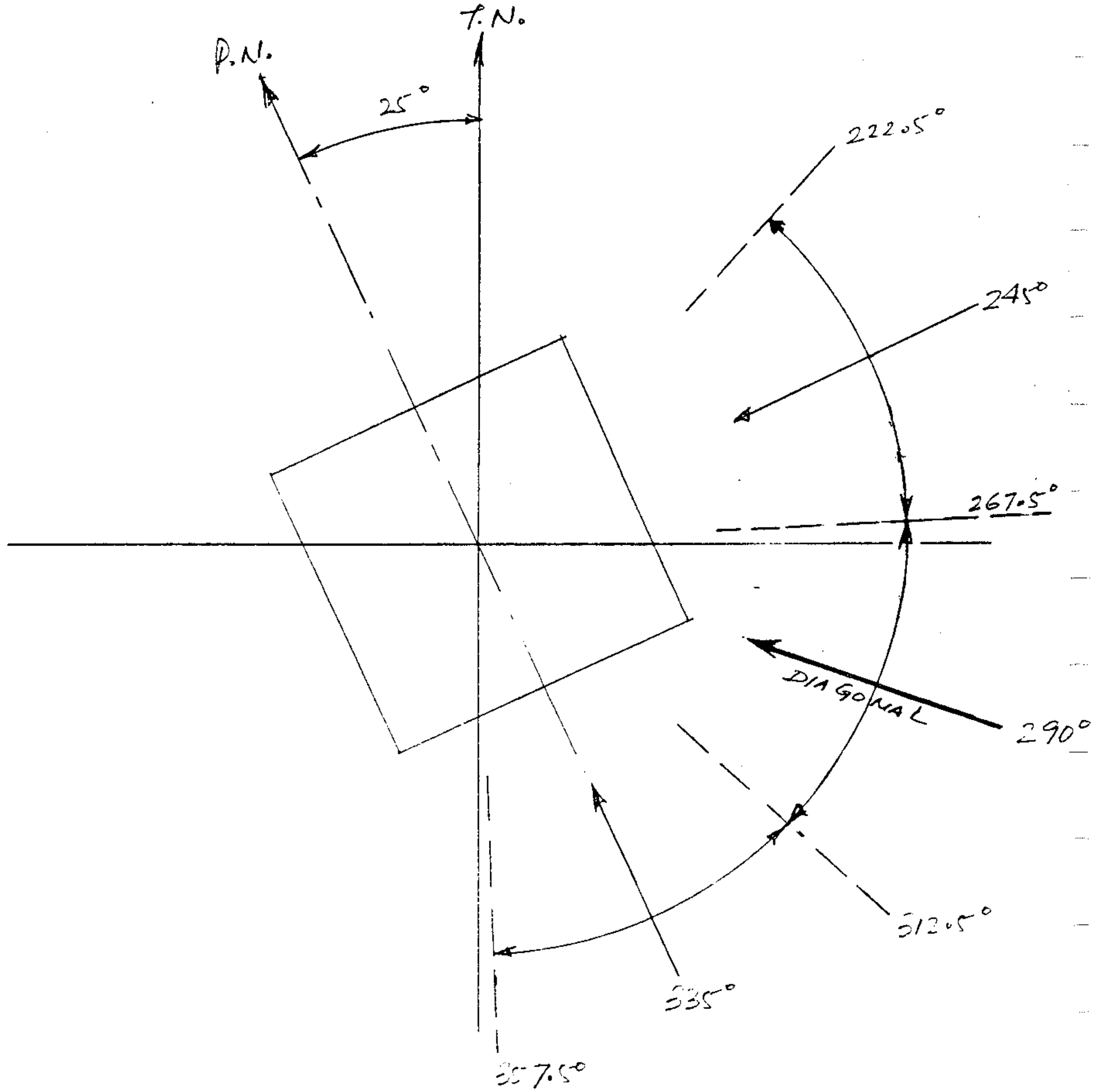
Platform ST130Q

PMB ENGINEERING

By R.K.A. Date 05/27/93 Checked by ______ Sheet No. ______

Project ANDREW J/P Job No. 295

Subject PLATFORM ST 130 'QTRS'

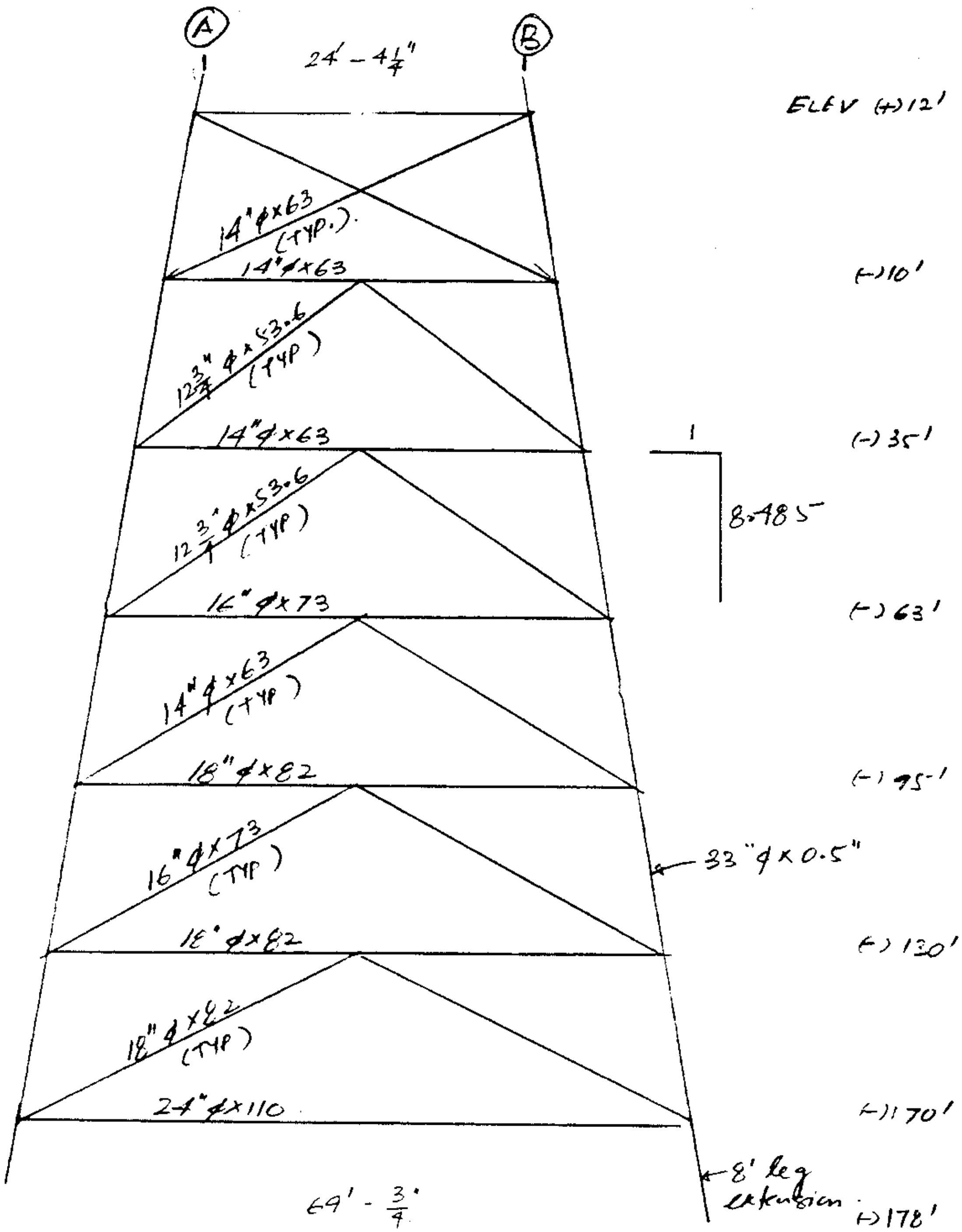


PMB ENGINEERING

By________ Date / / Checked by________ Sheet No.________

Project ANDREW Job No.________

Subject ST-180 Q

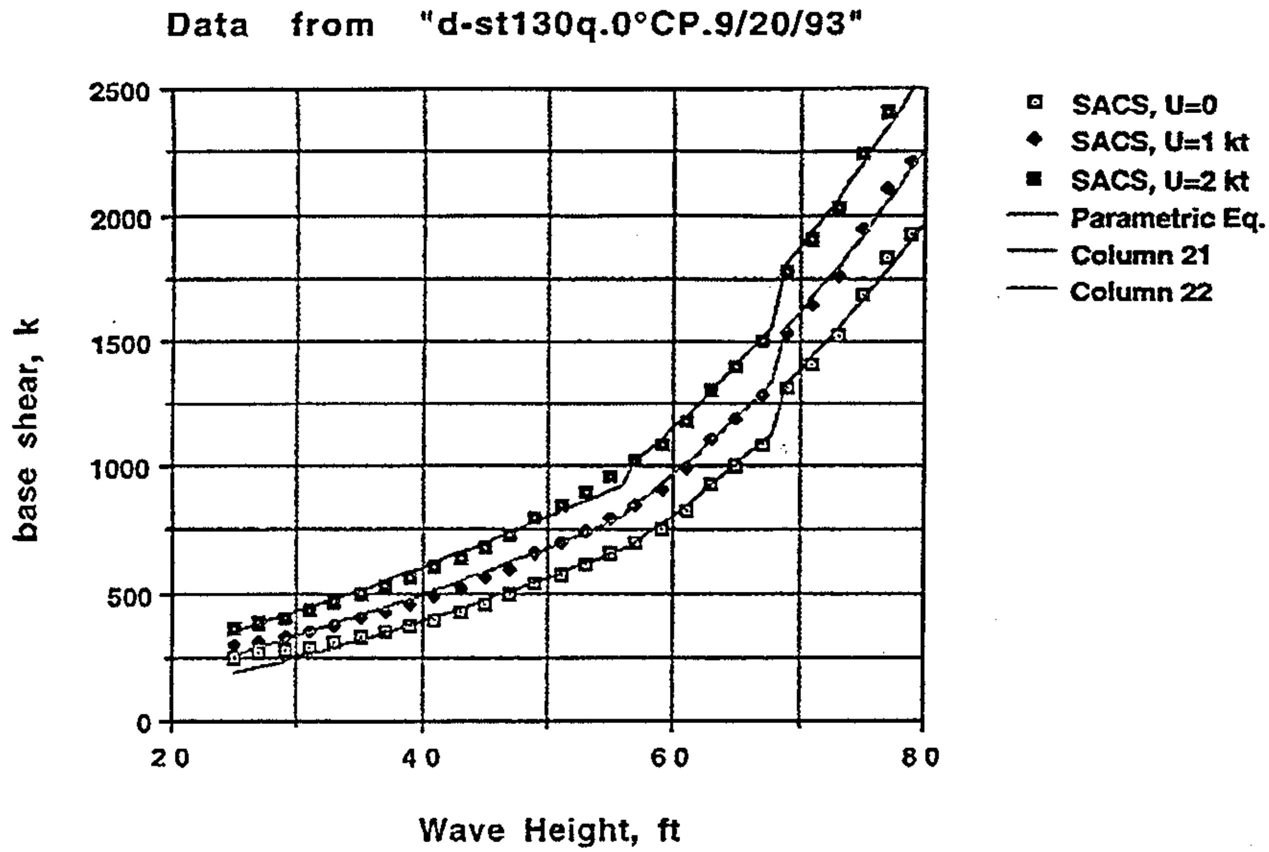


Notes:

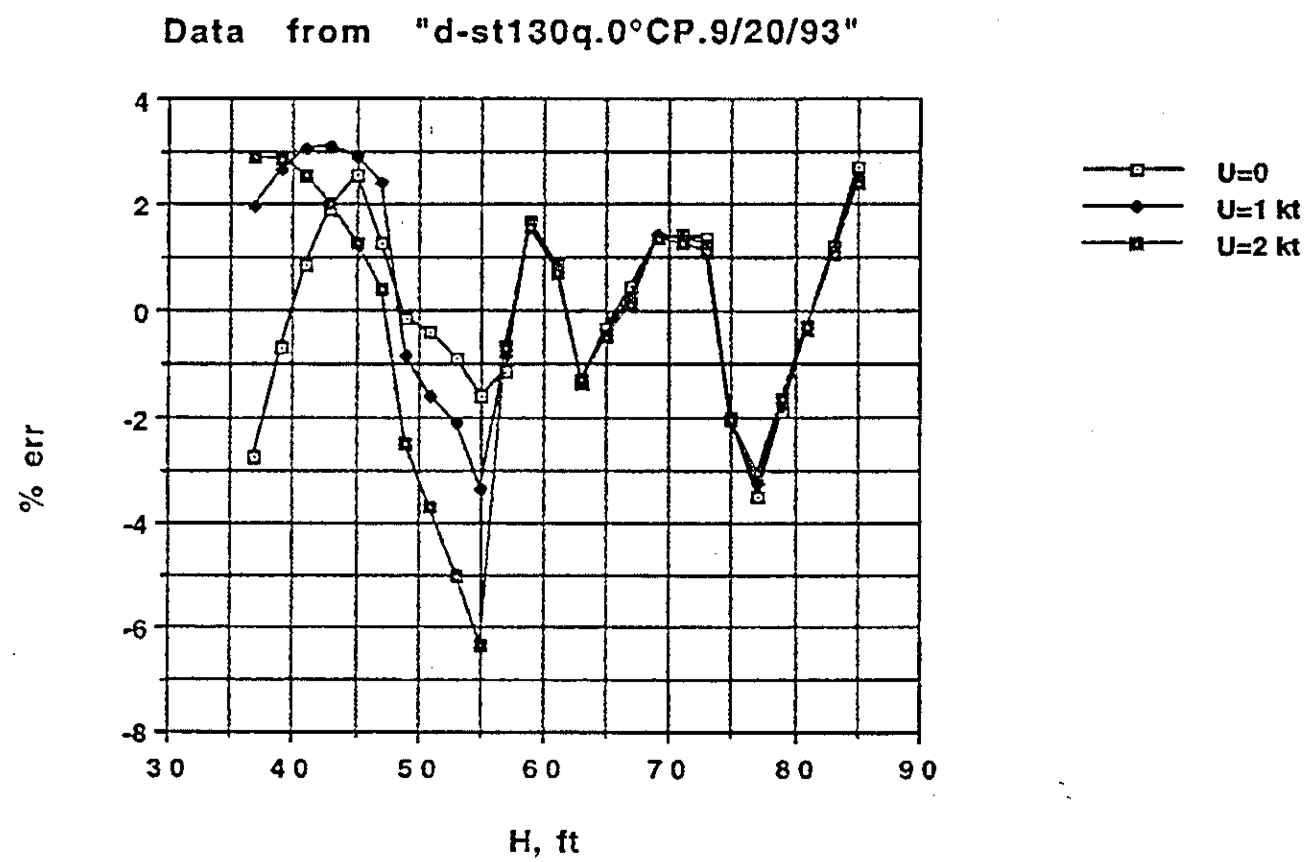
1. Brace size given as: Diameter x Unit weight in lb.
2. Drawn N.T.S.

C1, C2, & C3 for ST130QTR   Mon, Sep 20, 1993 11:51 PM

BRANCH	C1	C2	C3
Wave Direction = 0°	END-ON		
H <= 56 ft	1.053e+0	6.148e+0	1.602e+0
56 ft < H <= 68 ft	8.768e-3	4.147e+0	2.788e+0
H > 68 ft	1.973e-2	4.261e+0	2.625e+0
Wave Direction = 45°	DIAGONAL		
H <= 56 ft	1.186e+0	7.807e+0	1.559e+0
56 ft < H <= 68 ft	1.080e-2	5.118e+0	2.725e+0
H > 68 ft	1.768e-2	5.228e+0	2.628e+0
Wave Direction = 90°	BROADSIDE		
H <= 56 ft	1.042e+0	7.319e+0	1.595e+0
56 ft < H <= 68 ft	7.925e-3	4.689e+0	2.809e+0
H > 68 ft	1.559e-2	4.830e+0	2.668e+0



Data from "d-st130q.0°CP.9/20/93"
2500
2000
1500
1000
500
0
base shear, k
20
40
60
80
Wave Height, ft
SACS, U=0
SACS, U=1 kt
SACS, U=2 kt
Parametric Eq.
Column 21
Column 22



Data from "d-st130q.0°CP.9/20/93"
4
2
0
-2
-4
-6
-8
% err
30
40
50
60
70
80
90
H, ft
U=0
U=1 kt
U=2 kt

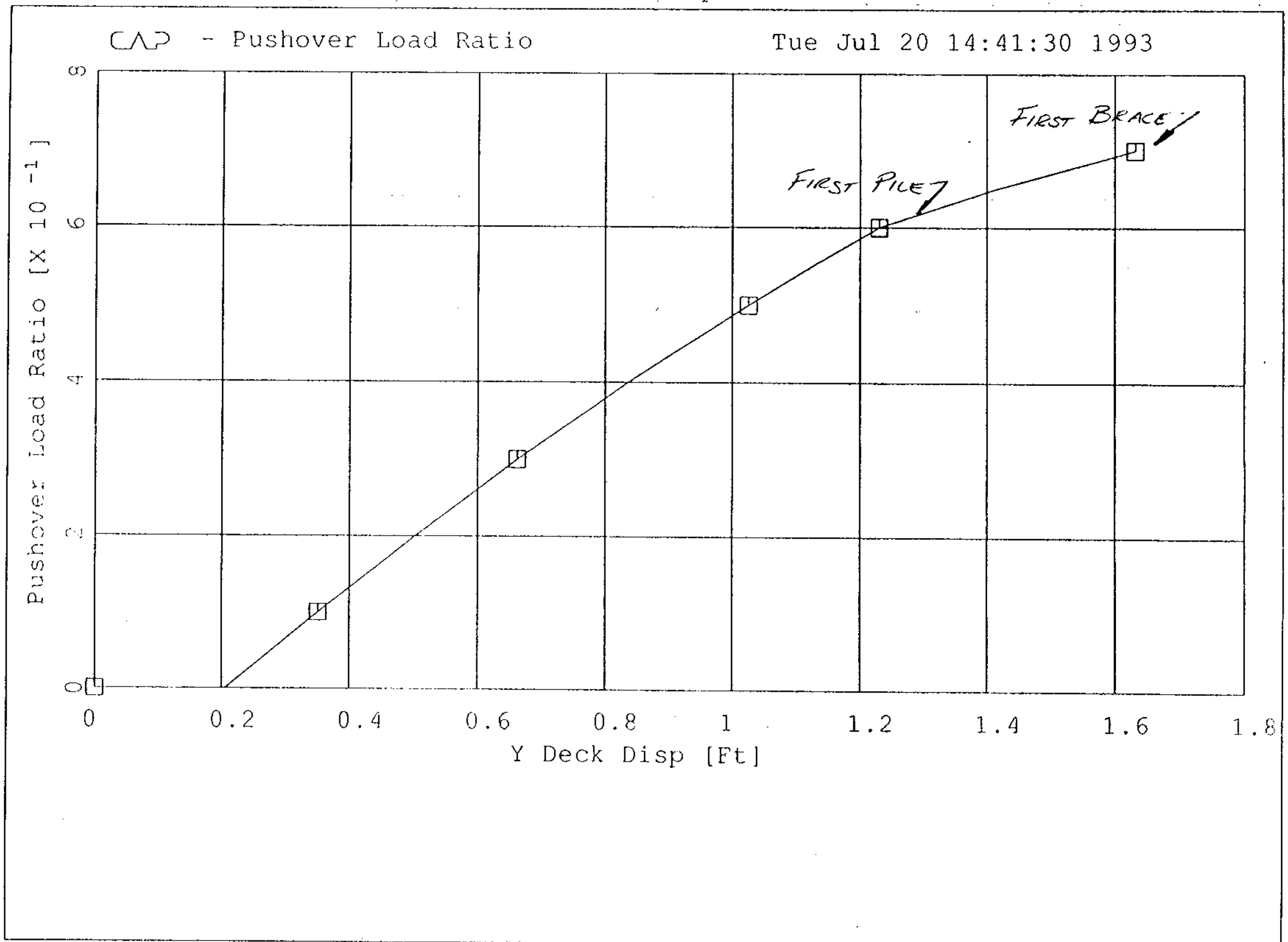
PLATFORM ST130Q

Water Depth = 170 ft.

This Platform Survived During Hurricane Andrew Loads

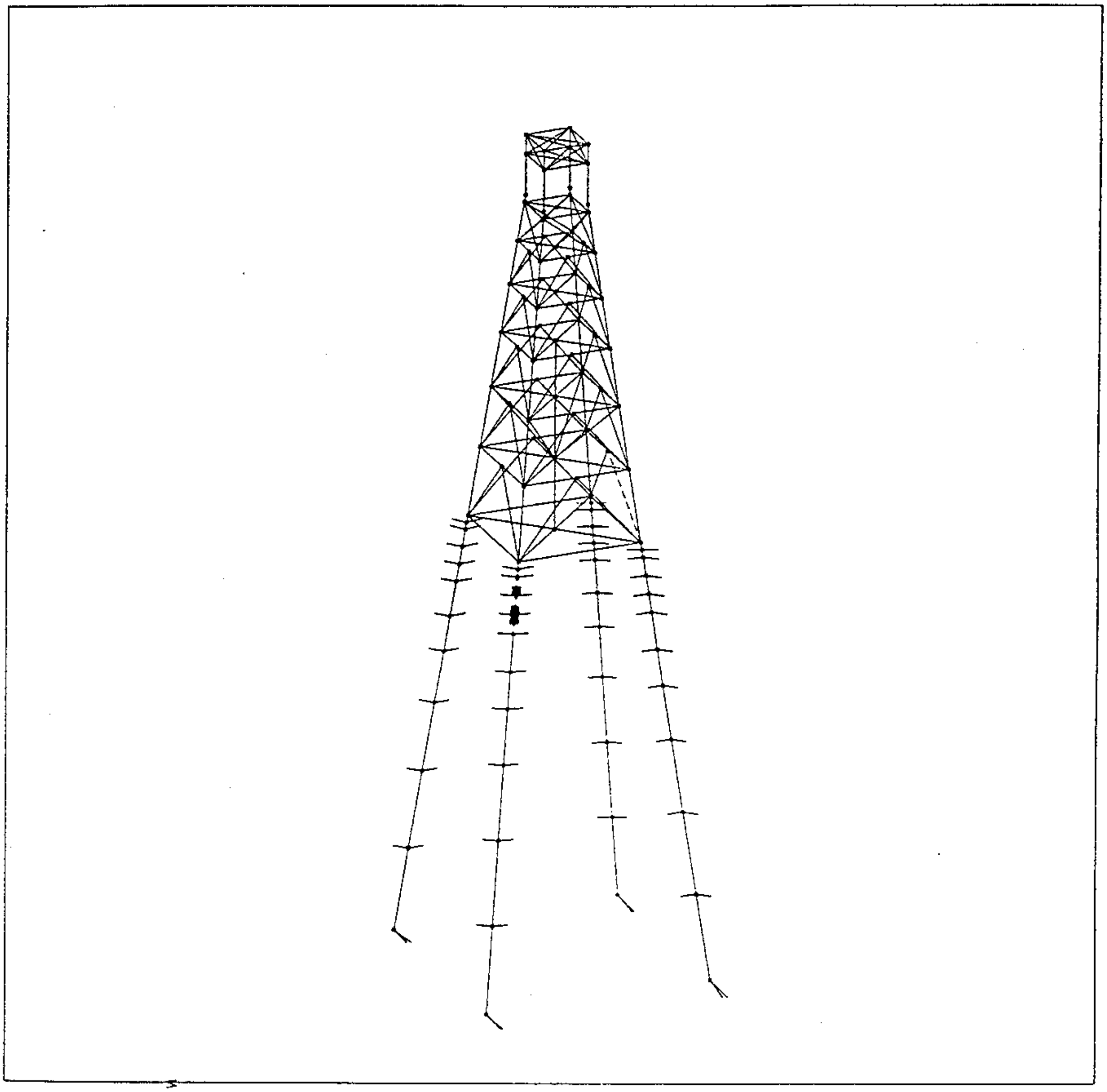
Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.693 * Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	U (knots)	C1	C2	C3	H+C2*U U in knots	BS (Kips)	Remarks	R median (Kips)
Broadside Direction:			**(222.5 deg. to 267.5 deg.)**										1420.00
1	254.1	19.14	32.404	10.194	7.54	0.53	1.042	7.319	1.595	36.259	320.00		
2	255.5	21.89	37.054	11.155	8.25	0.67	1.042	7.319	1.595	41.931	403.46		
3	255.6	24.43	41.366	11.579	8.57	0.85	1.042	7.319	1.595	47.602	493.95		
4	257.5	27.38	46.354	12.095	8.95	1.09	1.042	7.319	1.595	54.325	609.81		
5	261.9	31.20	52.821	12.963	9.59	1.37	1.042	7.319	1.595	62.851	769.44		
Diagonal Direction:			**(267.5deg. to 312.5 deg.)**										1265.00
6	272.6	35.17	59.549	13.724	10.16	1.65	0.0108	5.118	2.725	67.972	1062.99	Wave in Cellar Deck	
7	289.1	36.78	62.269	14.192	10.50	1.78	0.0108	5.118	2.725	71.359	1213.60	Wave in Cellar Deck	
8	306.0	34.10	57.735	13.615	10.08	1.53	0.0108	5.118	2.725	65.584	964.26		
End-On Direction:			**(312.5 deg. to 357.5deg.)**										1420.00
9	313.3	31.65	53.584	12.700	9.40	1.15	1.053	6.148	1.602	60.659	756.18		
10	316.9	29.54	50.020	12.100	8.95	0.74	1.053	6.148	1.602	54.588	638.64		
11	320.0	28.04	47.477	11.766	8.71	0.38	1.053	6.148	1.602	49.821	551.67		
12	322.9	26.19	44.332	11.308	8.37	0.17	1.053	6.148	1.602	45.375	474.94		
13	326.0	24.32	41.172	10.864	8.04	0.00	1.053	6.148	1.602	41.172	406.46		
14	327.9	22.87	38.715	10.668	7.89	0.00	1.053	6.148	1.602	38.715	368.31		
15	329.4	21.54	36.471	10.537	7.80	0.00	1.053	6.148	1.602	36.471	334.70		
16	330.1	20.28	34.340	10.313	7.63	0.00	1.053	6.148	1.602	34.340	303.93		
17	330.5	19.00	32.160	9.929	7.35	0.00	1.053	6.148	1.602	32.160	273.62		

Project: ChevST130Q Model: push225 Version: 1



Chevron ST130Q 22.5 Deg (65' wave)

CAPACITY = 1265k
APPLIED LOAD = 1807
FIRST PILE @ 1118



CAP

Chevron ST130Q 22.5 Deg (65'wave) Fcap=1265 K

Project: ChevST130Q Model: push225 Version: 1

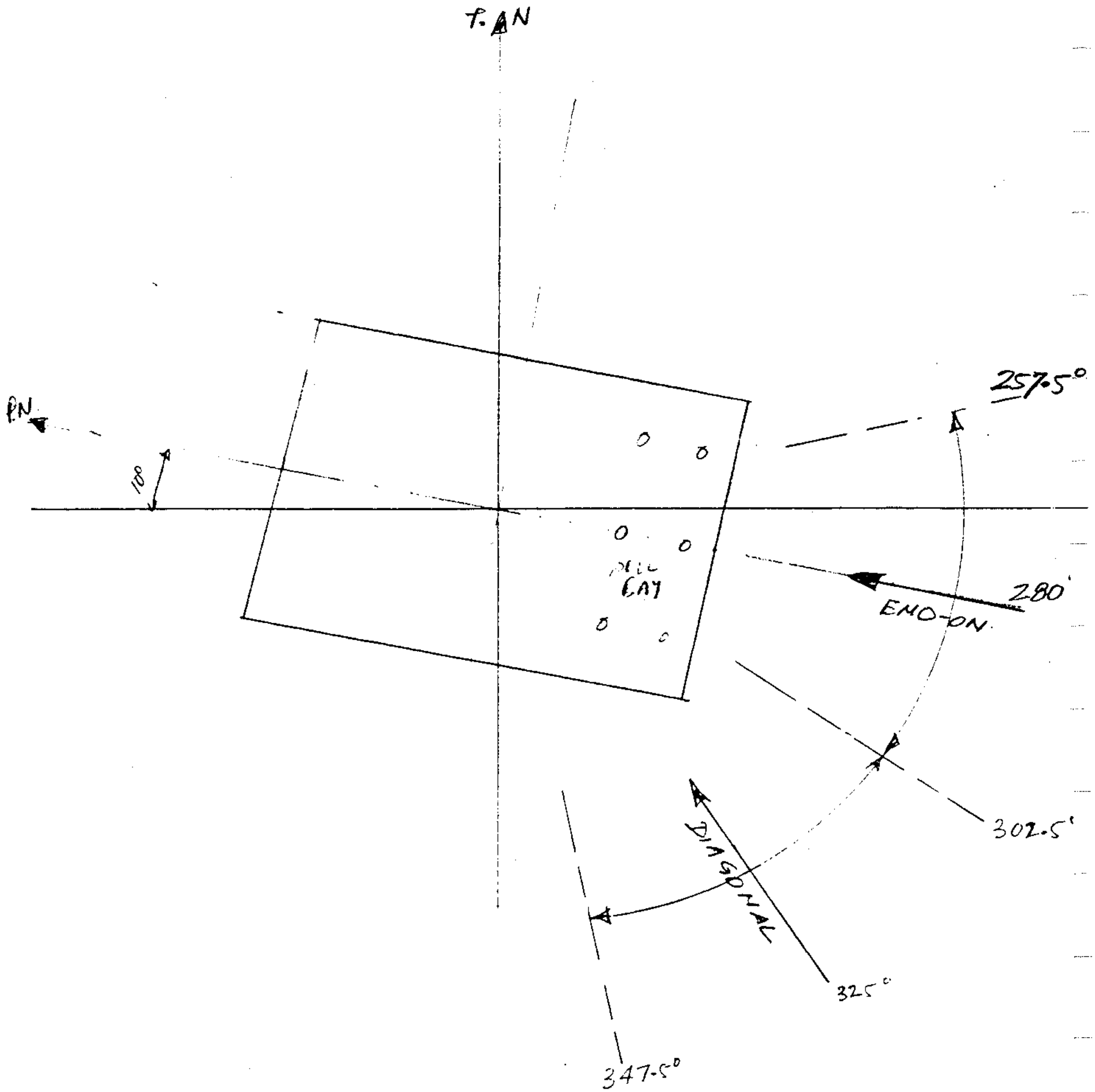
Platform ST134W

PMB ENGINEERING

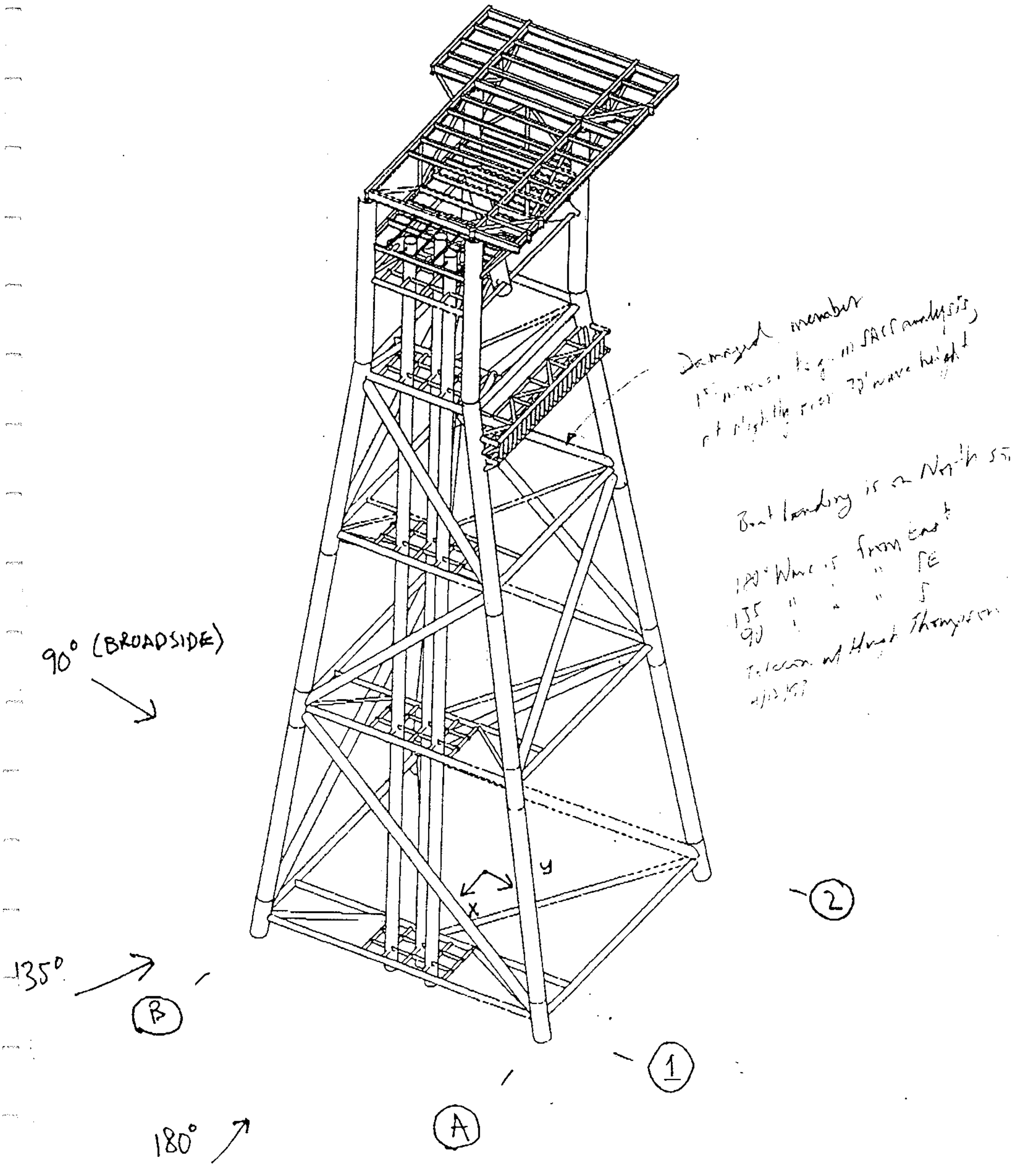
By RKA Date 05/25/93 Checked by ________ Sheet No. ________

Project ANDREW JIP Job No. 255

Subject PLATFORM ST134W.



SOUTH TIMBALIER 134W 4-PILE 6-WELL PLATFORM

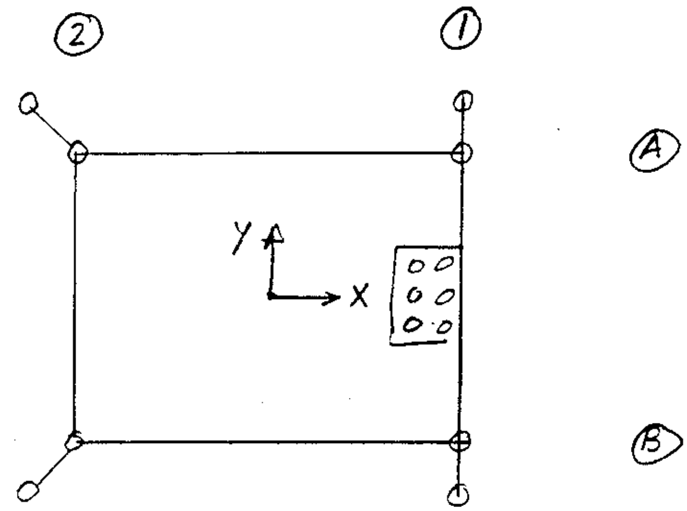


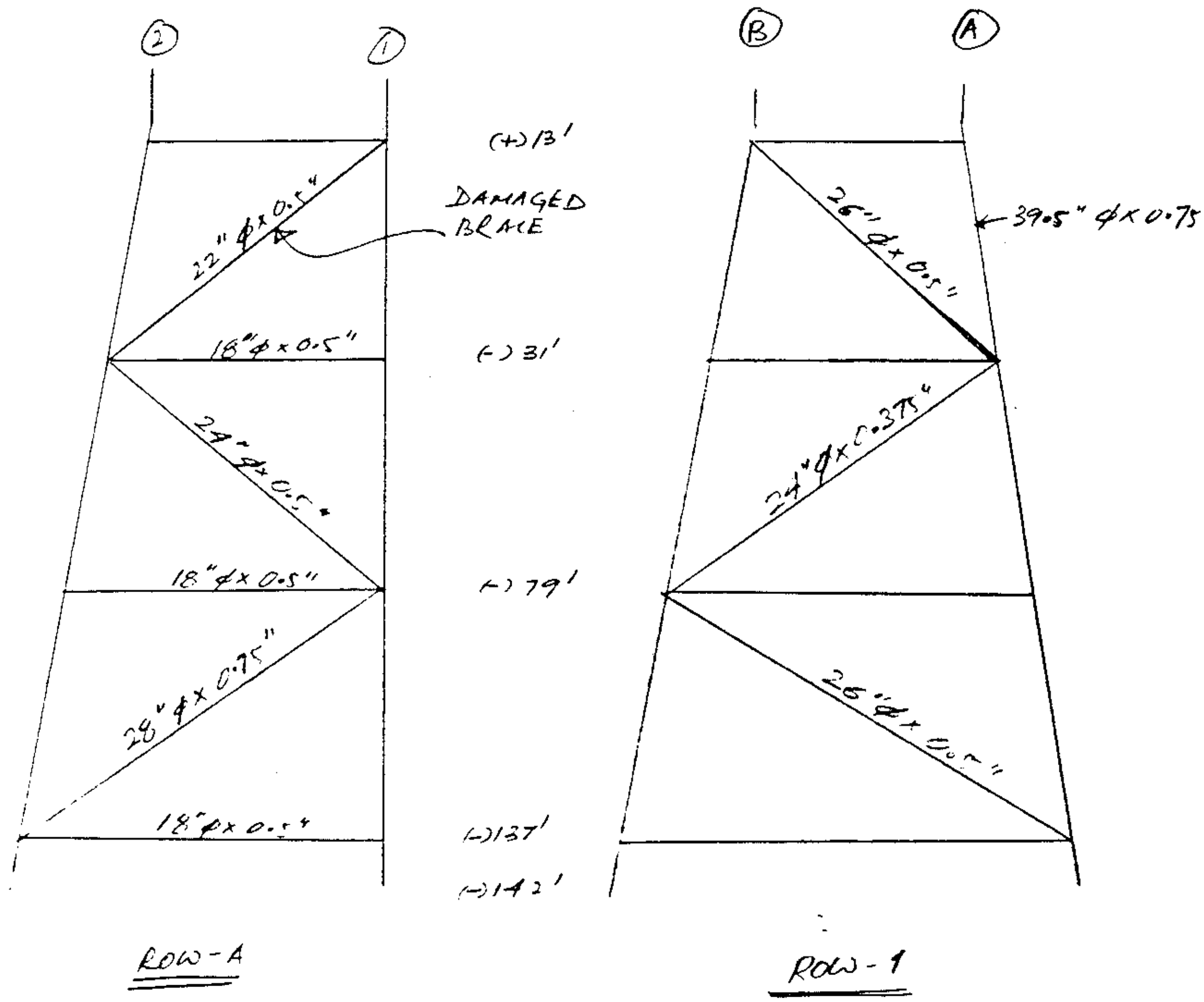
PMB ENGINEERING

By RKA Date / / Checked by ______ Sheet No. ______

Project ANDREW JIP Job No. 295-

Subject PLATFORM ST 134 D

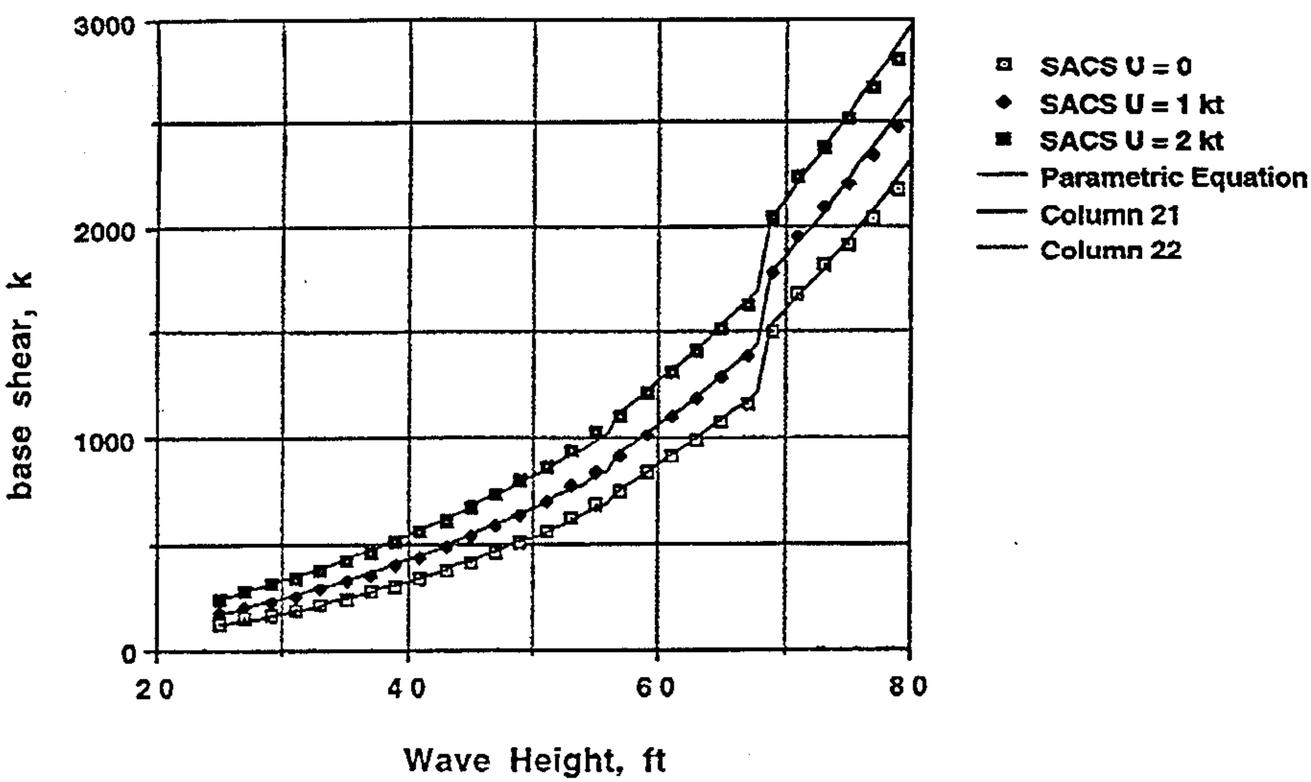




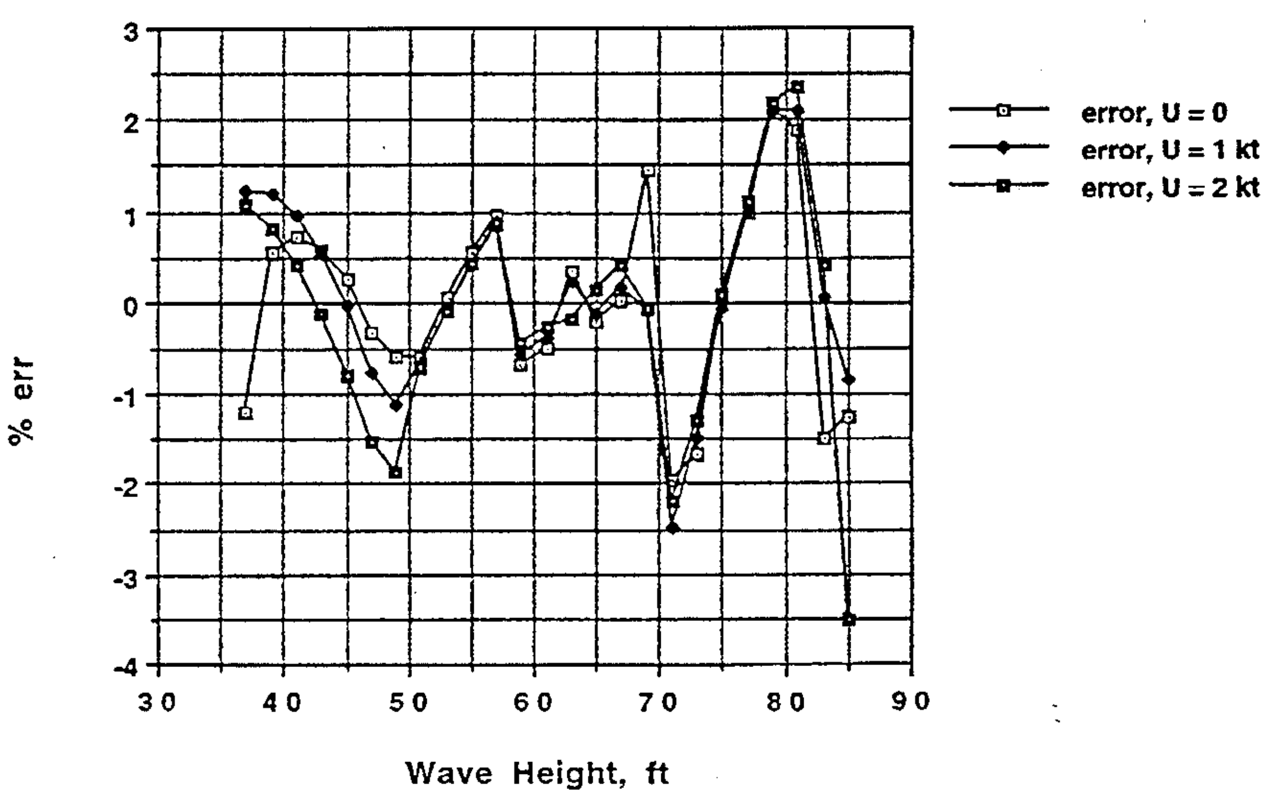
C1,C2,C3 for ST134W,9/93.rs7 Tue, Sep 21, 1993 1:56 AM

BRANCH	C1	C2	C3
Wave Direction = 180°	END-ON		
H <= 50 ft	6.968e-2	5.129e+0	2.285e+0
50 ft < H <= 68 ft	1.369e-2	4.421e+0	2.700e+0
H > 68 ft	1.217e-2	3.853e+0	2.772e+0
Wave Direction = 225°	DIAGONAL		
H <= 50 ft	9.095e-2	6.160e+0	2.222e+0
50 ft < H <= 68 ft	1.414e-2	5.205e+0	2.697e+0
H > 68 ft	1.129e-2	4.720e+0	2.773e+0
Wave Direction = 90°	BROADSIDE		
H <= 50 ft	9.053e-2	5.803e+0	2.243e+0
50 ft < H <= 68 ft	1.293e-2	4.817e+0	2.739e+0
H > 68 ft	1.038e-2	4.396e+0	2.812e+0

Data from "d-st134w.180°.CP.9/93.rs7"



Data from "d-st134w.180°.CP.9/93.rs7"



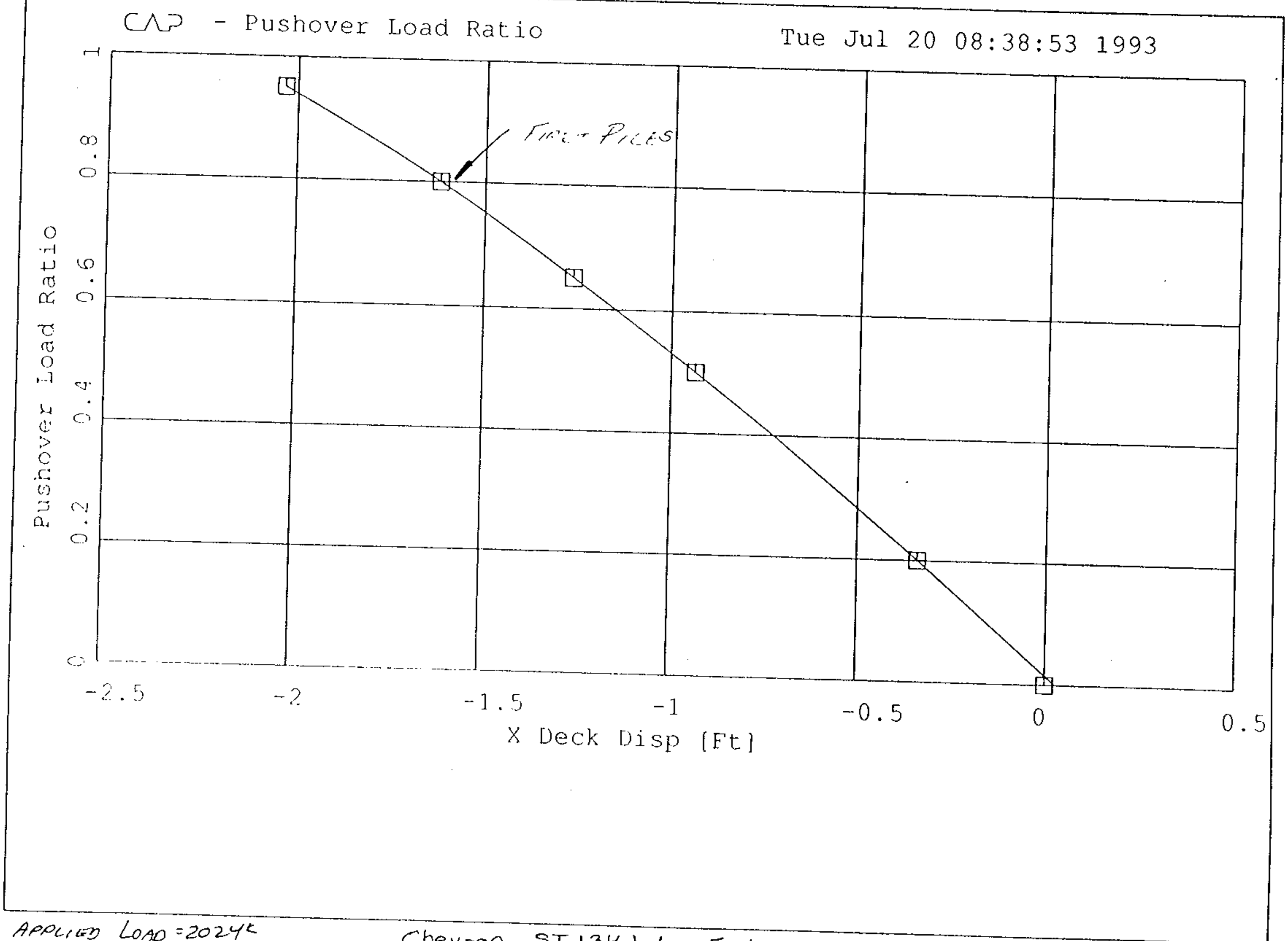
PLATFORM ST134W

This Platform was Damaged During Hurricane Andrew Loads

Water Depth = 137 ft.

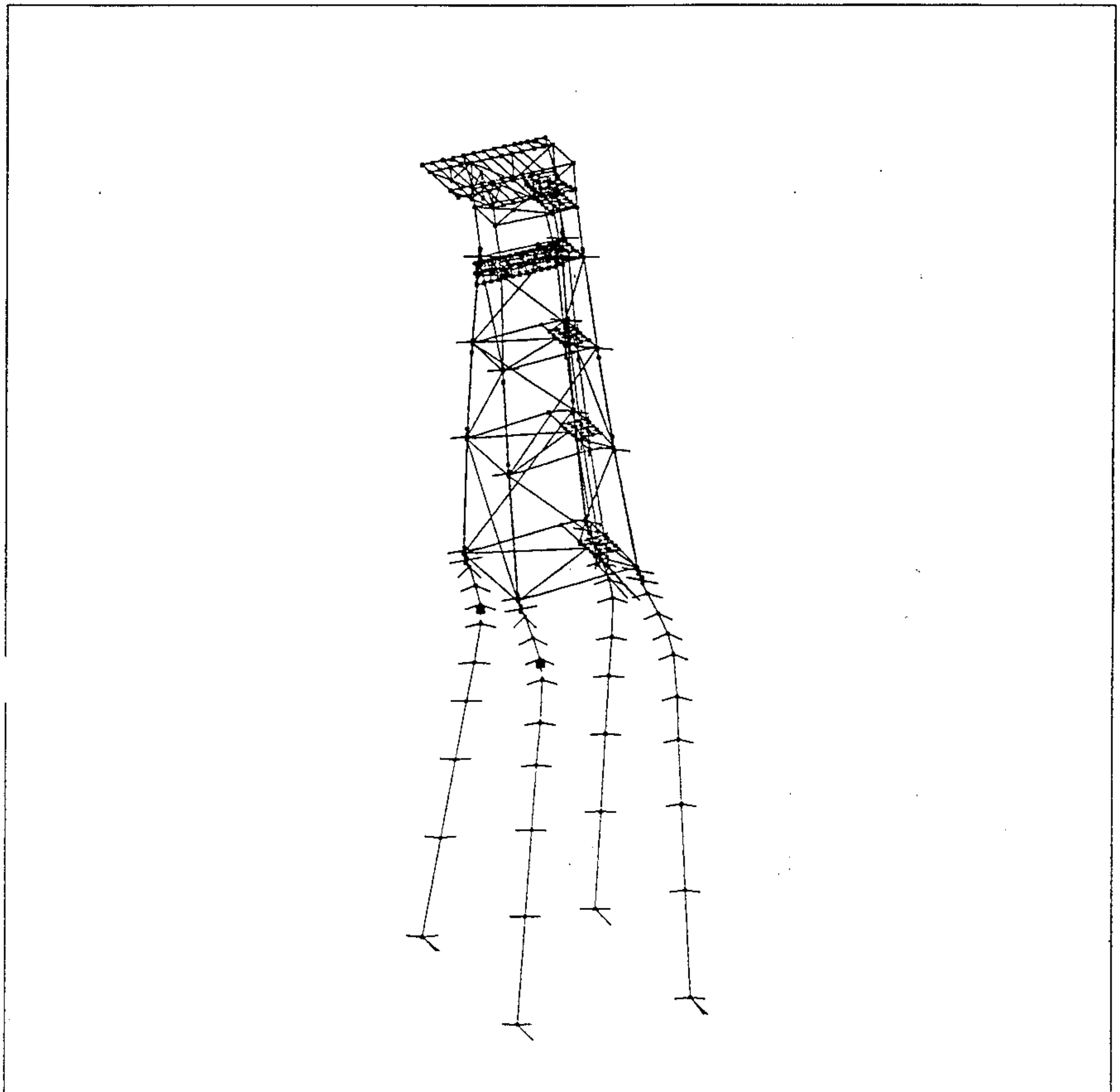
Storm Hour	Wave Direction (Degree)	H max = Hs * 1.683 (ft.)	Hs Hs =Hm/ 1.73 (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	U (knots)	C1	C2	C3	H+C2*U	BS (KIps)	Remarks	R median (KIps)
End-On Direction			(257.5 deg. to 302.5 deg.)										1923.00
5	258.3	49.882	29.64	12.724	9.42	1.53	0.06968	5.129	2.285	57.737	737.95		
6	268.2	57.029	33.89	13.669	10.12	1.87	0.01369	4.421	2.7	65.281	1087.23		
7	286.4	60.854	36.16	14.447	10.69	2.04	0.01369	4.421	2.7	69.886	1306.96		
Diagonal Direction			(302.5 deg. to 347.5 deg.)										1915.00
8	305.8	56.288	33.45	13.846	10.25	1.77	0.01414	5.205	2.697	65.483	1118.29		
9	314.2	52.277	31.06	12.947	9.58	1.34	0.01414	5.205	2.697	59.272	854.74		
10	317.7	48.762	28.97	12.245	9.06	0.92	0.09095	6.16	2.222	54.426	654.27		
11	321.6	46.278	27.50	11.763	8.70	0.52	0.09095	6.16	2.222	49.496	529.83		
12	324.9	43.410	25.79	11.401	8.44	0.31	0.09095	6.16	2.222	45.294	435.04		
13	327.4	40.412	24.01	10.924	8.08	0.12	0.09095	6.16	2.222	41.124	351.02		
14	329.2	37.927	22.54	10.673	7.90	0.02	0.09095	6.16	2.222	38.022	294.88		
15	330.8	35.681	21.20	10.516	7.78	0.00	0.09095	6.16	2.222	35.681	256.04		
16	331.7	33.620	19.98	10.246	7.58	0.00	0.09095	6.16	2.222	33.620	224.34		
17	331.7	31.409	18.66	9.871	7.30	0.00	0.09095	6.16	2.222	31.409	192.87		

Project: ChevST134W Model: pushx Version: 1



Chevron ST 134 W End On

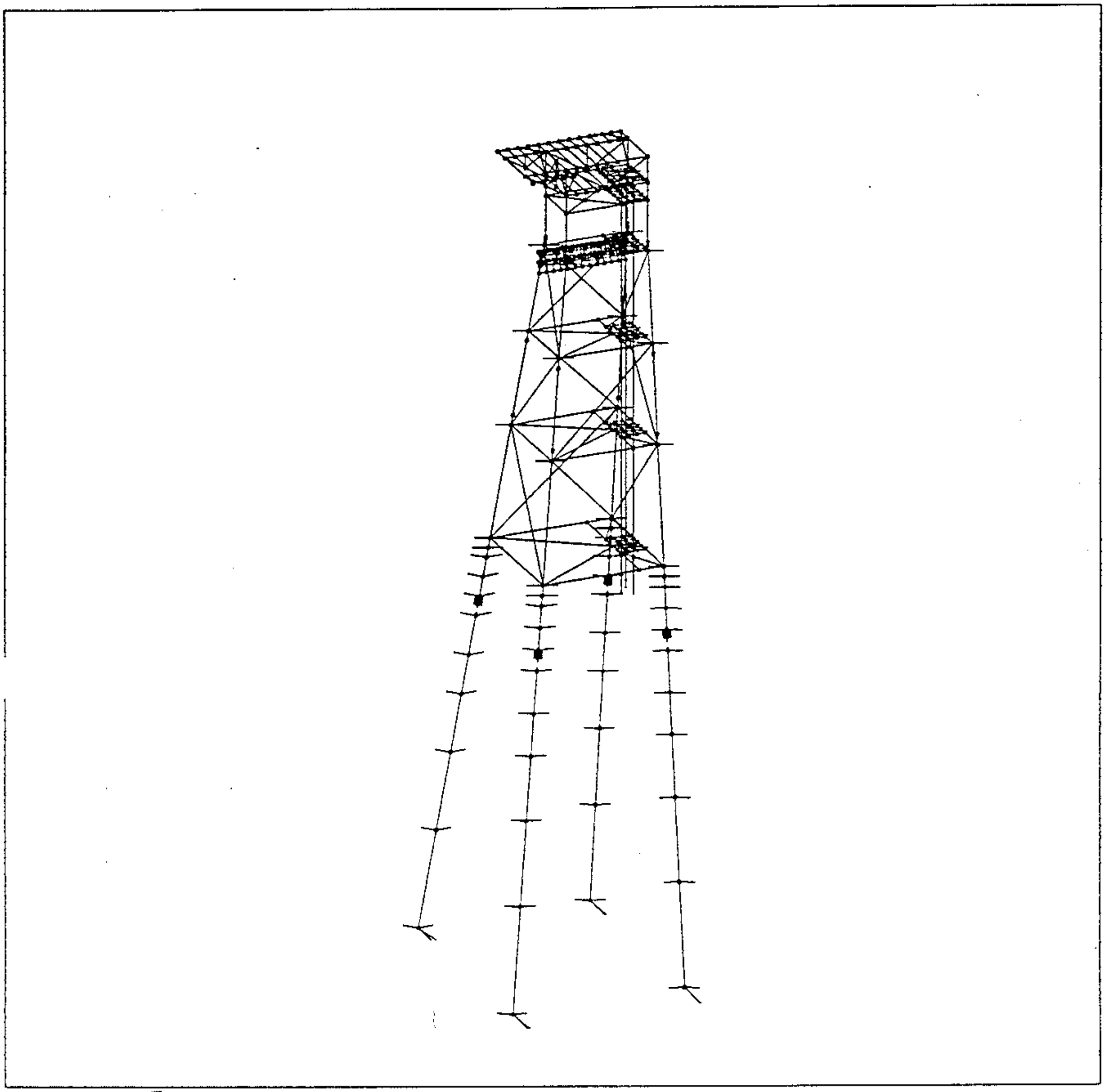
APPLIED LOAD = 2024k
CAPACITY = 1923k
FIRST PILE @ 1620k



CAP

Chevron ST134W End On(70'wave)1st pile = 1620K

Project: ChevST134W Model: pushx Version: 1



CAP Chevron ST134W End On (70'wave) Fcap = 1923 K

Project: ChevST134W Model: pushx Version: 1

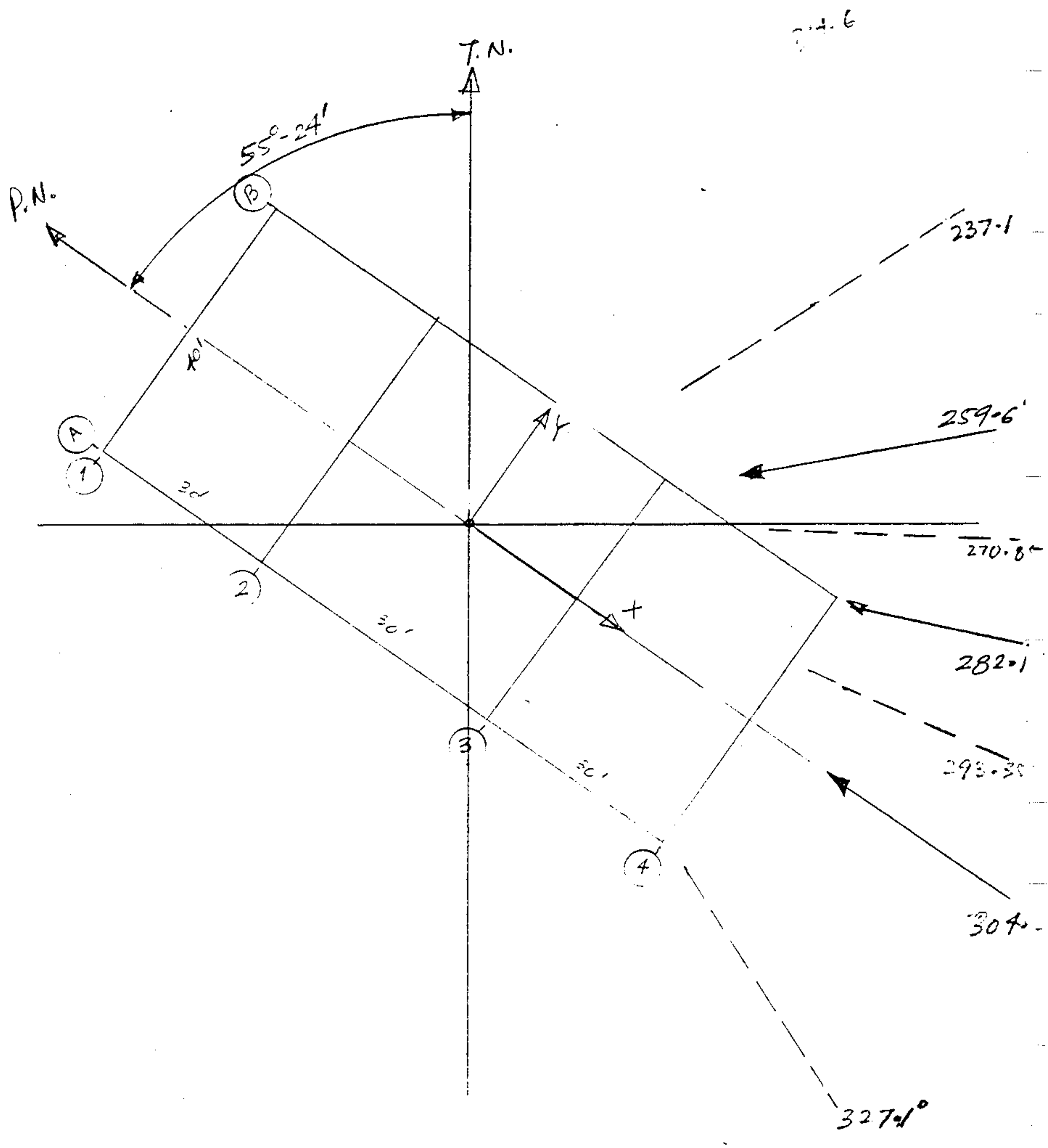
Platform WD 90A

PMB ENGINEERING

By RKA Date 06/15/93 Checked by ______ Sheet No. ______

Project ANDREW JIP Job No. 295

Subject PLATFORM WD90 A

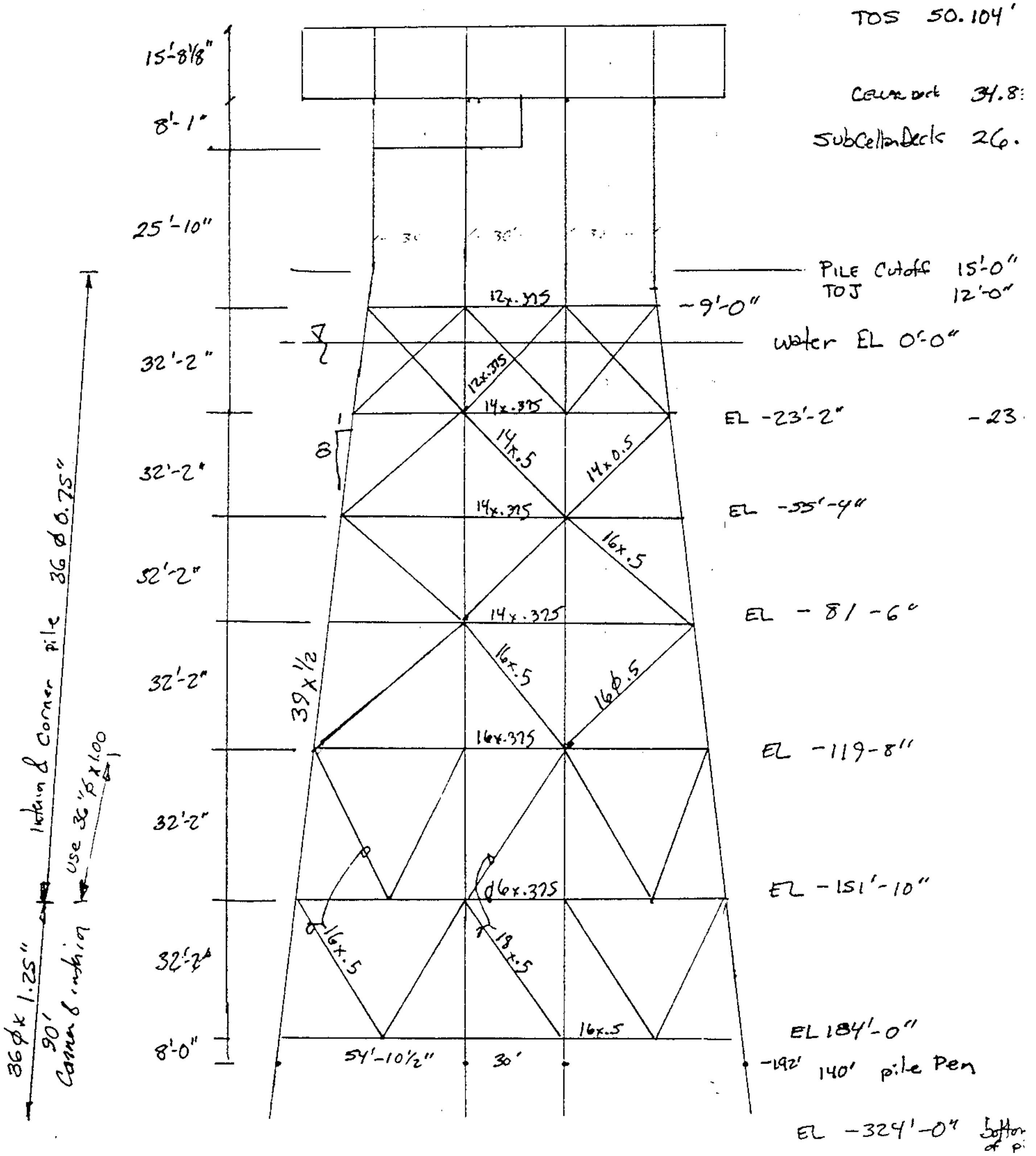


PMB ENGINEERING

By: ML Date: 6/ /72 Checked by: Sheet No.:

Project: ANDREW Job No.: 295

Subject: WD-90



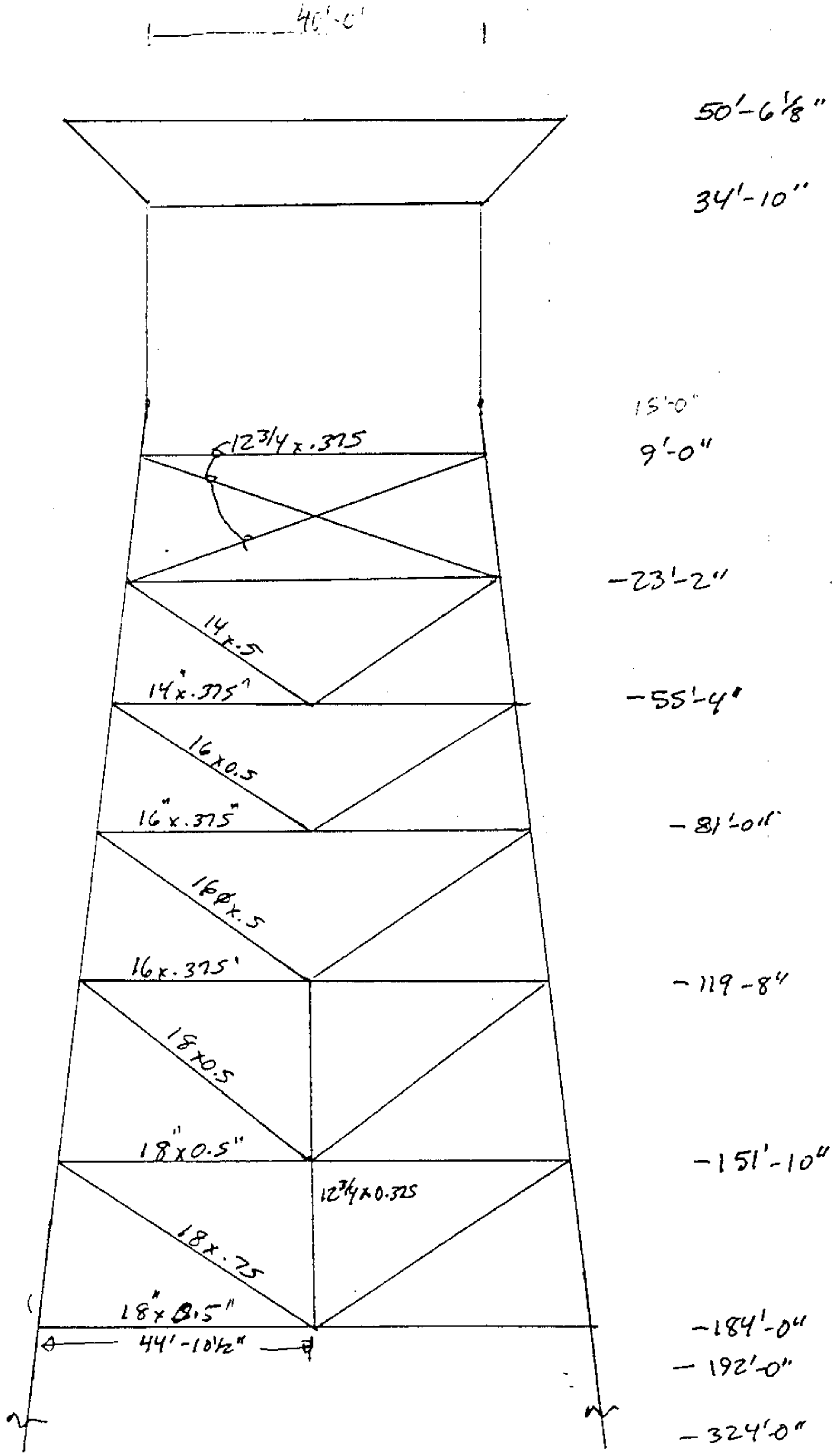
Rows A & B.

PMB ENGINEERING

By: ML Date: 6/8/93 Checked by: ____ Sheet No. ____

Project: Archer Job No. 295

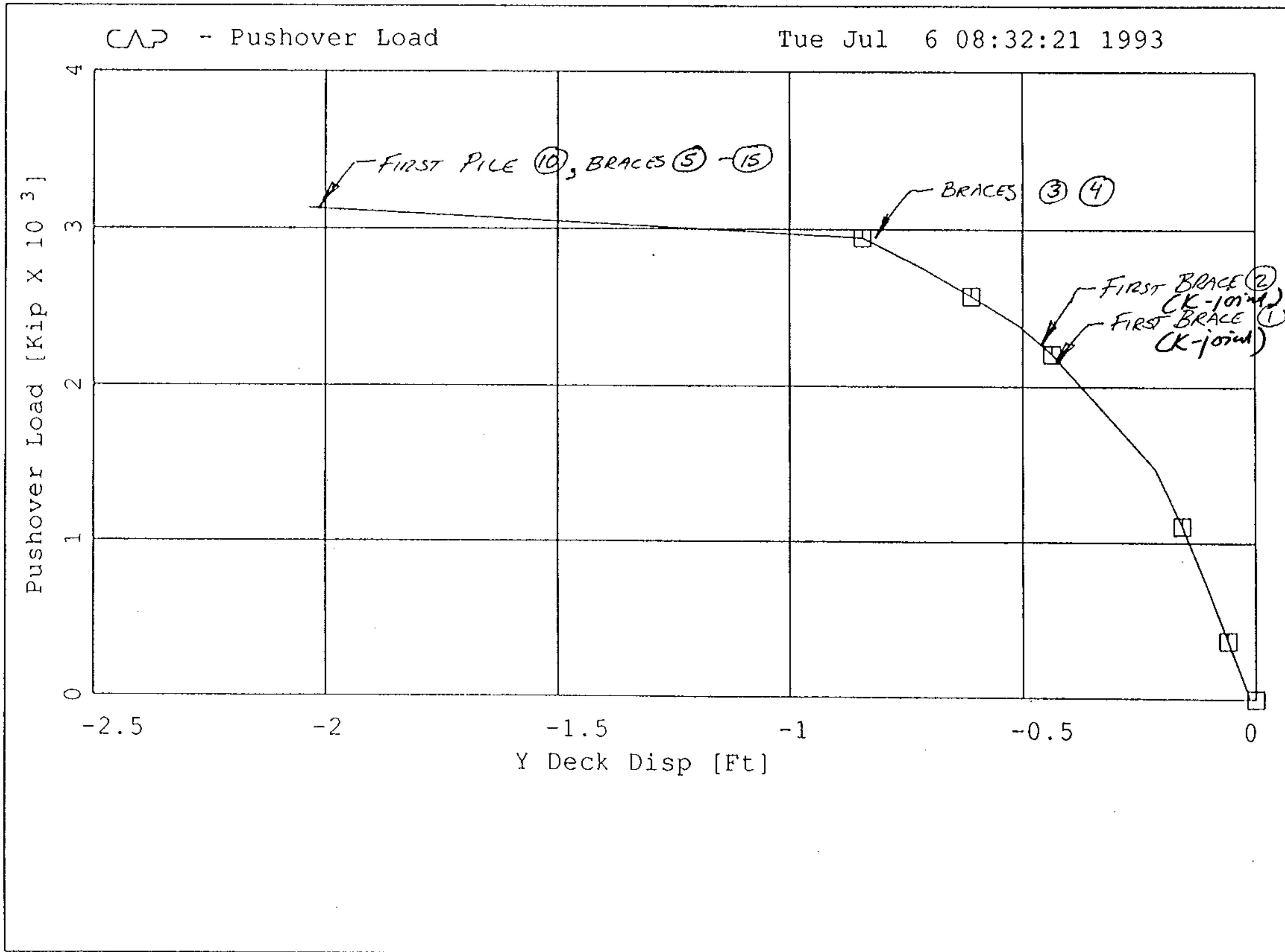
Subject: WD 90 A



ROWS 1–4

PLATFORM WD90A				This Platform Survived Against Hurricane Andrew Loads									
Water Depth =		184	ft.										
Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.7650 * Hs (ft.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	U with blockage (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Diagonal-1 Direction			(237.1 deg. to 270.85 deg.)										2988.00
1	263.07	13.89	24.512	6.32	0.38367	3.43957	2.11676	0.63	0.53	26.347	390.23		
2	266.07	15.65	27.614	6.58	0.38367	3.43957	2.11676	0.76	0.65	29.838	507.78		
3	268.88	17.44	30.775	6.78	0.38367	3.43957	2.11676	0.92	0.79	33.478	647.90		
Diagonal-2 Direction			(270.85 deg. to 293.35 deg.)										3130.00
4	272.85	19.50	34.424	8.43	0.51359	3.31683	2.05209	1.12	0.96	37.596	876.89		
5	275.14	21.97	38.777	8.65	0.51359	3.31683	2.05209	1.39	1.18	42.703	1138.84		
6	279.11	23.74	41.900	9.78	0.51359	3.31683	2.05209	1.70	1.45	46.698	1368.27		
7	284.61	25.38	44.798	9.77	0.51359	3.31683	2.05209	2.00	1.70	50.432	1602.20		
8	290.19	27.07	47.782	9.81	0.51359	3.31683	2.05209	2.27	1.93	54.171	1855.50		
End-On Direction			(293.35 deg. to 327.1 deg.)										3267.00
9	297.10	28.38	50.096	9.81	0.04854	2.78993	2.64893	2.51	1.75	54.989	1976.92	Wave in Cellar Deck	
10	303.36	28.63	50.531	9.76	0.04854	2.78993	2.64893	2.56	1.79	55.537	2029.59	Wave in Cellar Deck	
11	308.98	27.57	48.657	9.14	0.04854	2.78993	2.64893	2.38	1.67	53.305	1820.63		
12	312.23	26.53	46.822	8.85	0.04854	2.78993	2.64893	2.08	1.46	50.885	1609.81		
13	314.86	25.65	45.272	8.76	0.04854	2.78993	2.64893	1.81	1.26	48.799	1440.84		
14	317.31	24.36	42.987	8.56	0.31577	3.62615	2.15301	1.62	1.14	47.108	1263.46		
15	319.81	22.82	40.283	8.14	0.31577	3.62615	2.15301	1.52	1.07	44.150	1098.81		
16	322.70	21.60	38.124	7.92	0.31577	3.62615	2.15301	1.41	0.99	41.712	972.30		
17	325.01	20.77	36.667	7.87	0.31577	3.62615	2.15301	1.27	0.89	39.881	882.74		
18	327.35	20.04	35.374	7.85	0.31577	3.62615	2.15301	1.16	0.81	38.308	809.47		

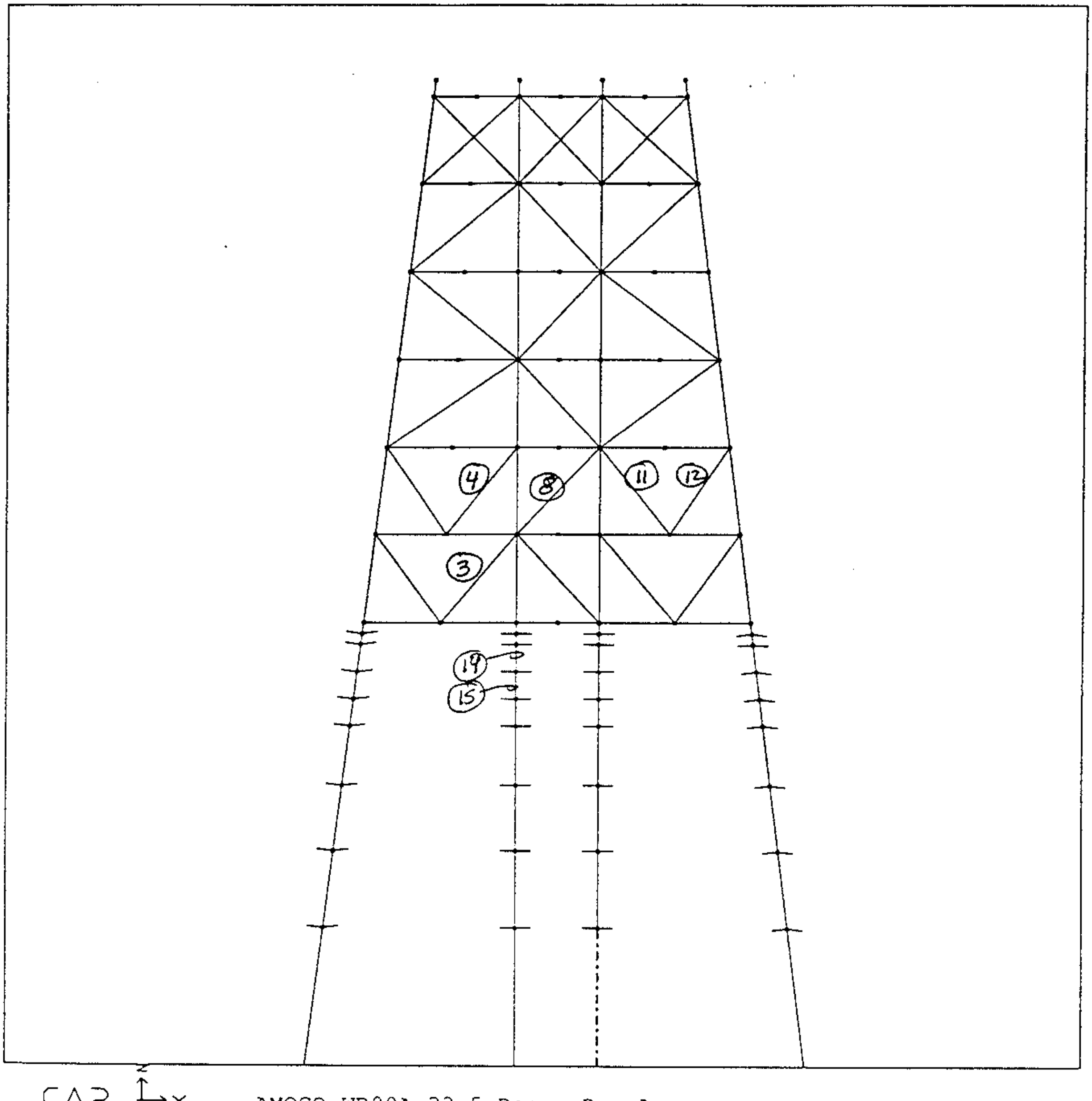
Project: AMOCO Model: push225 Version: 1



Capacity: 3130 kips

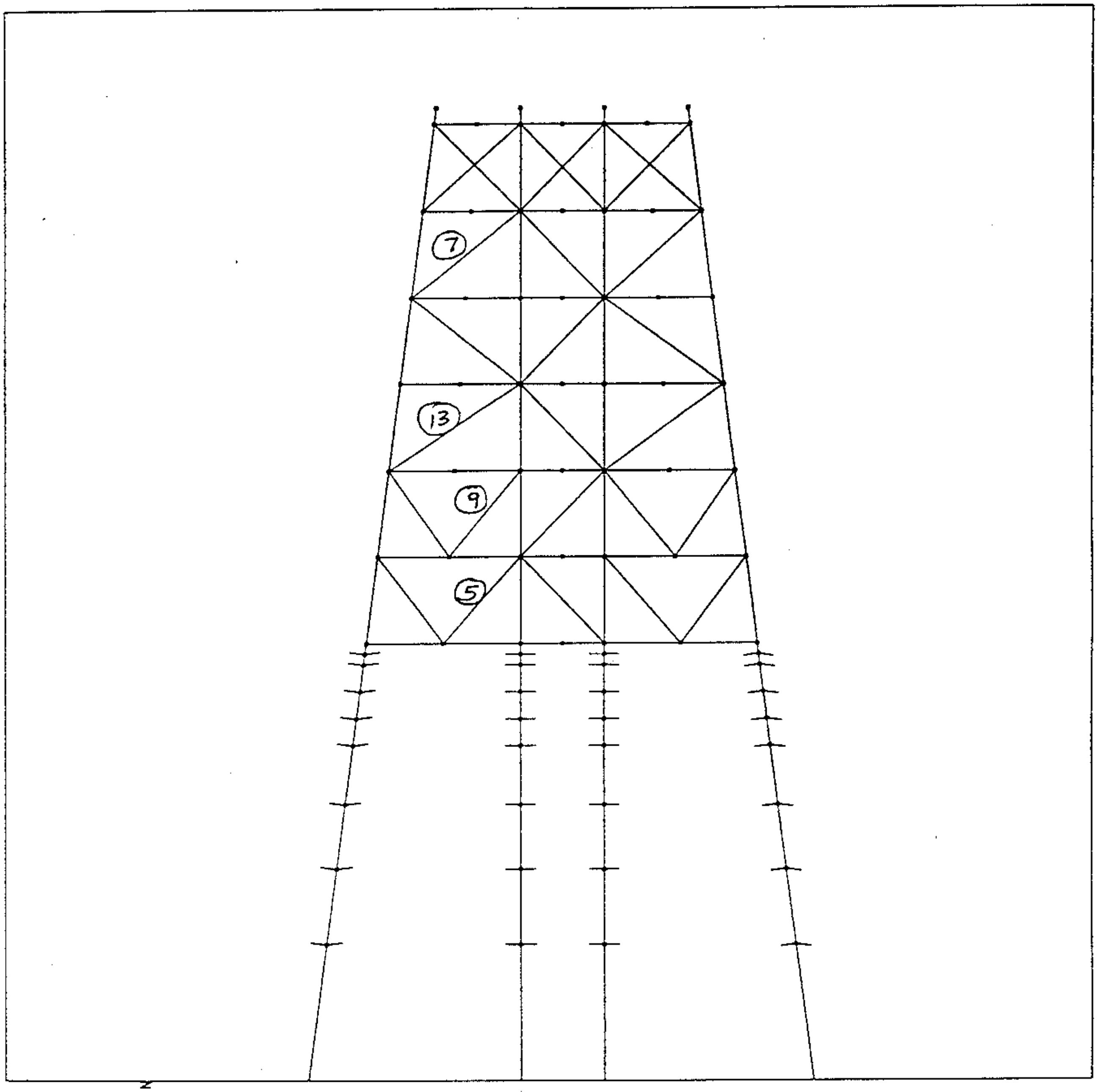
AMOCO WD90A 22.5 Deg

First K-Joint: 2393



AMOCO WD90A 22.5 Deg: Row A

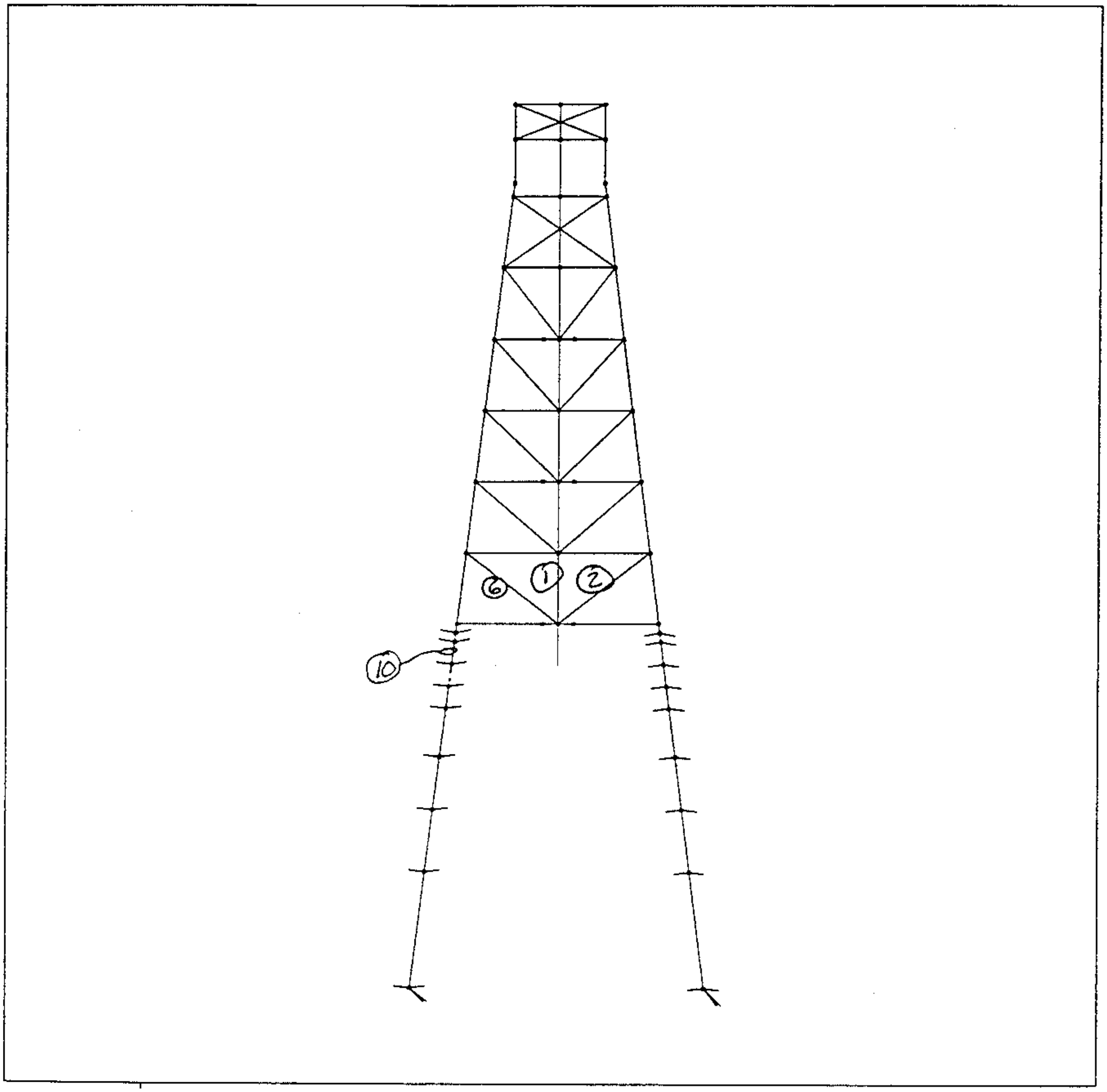
Project: AMOCO Model: push225 Version: 1



CAP

AMOCO WD90A 22.5 Deg: Row B

Project: AMOCO Model: push225 Version: 1



CAP ⊢-

AMOCO WD90A 22.5 Deg: Row 1

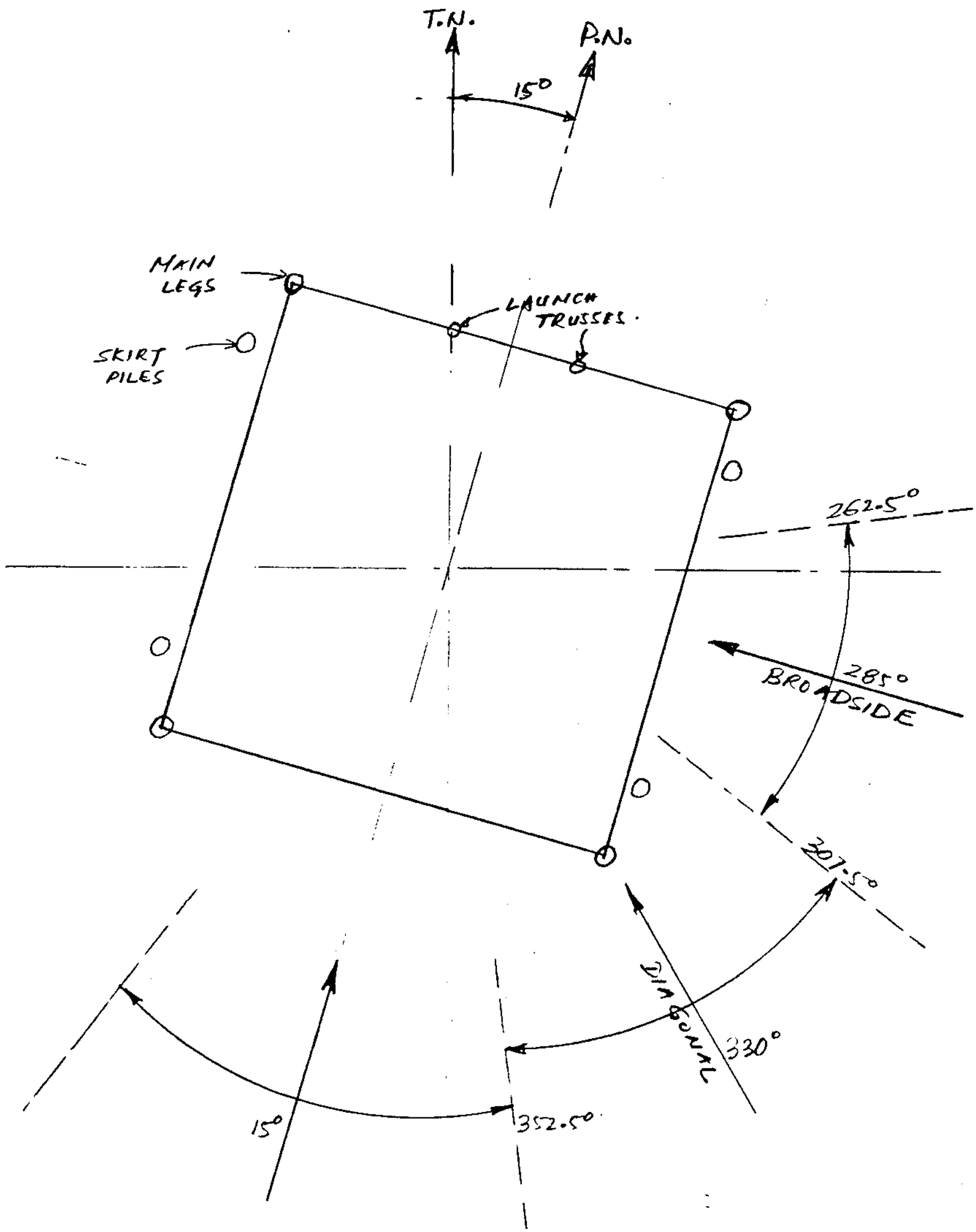
Project: AMOCO Model: push225 Version: 1

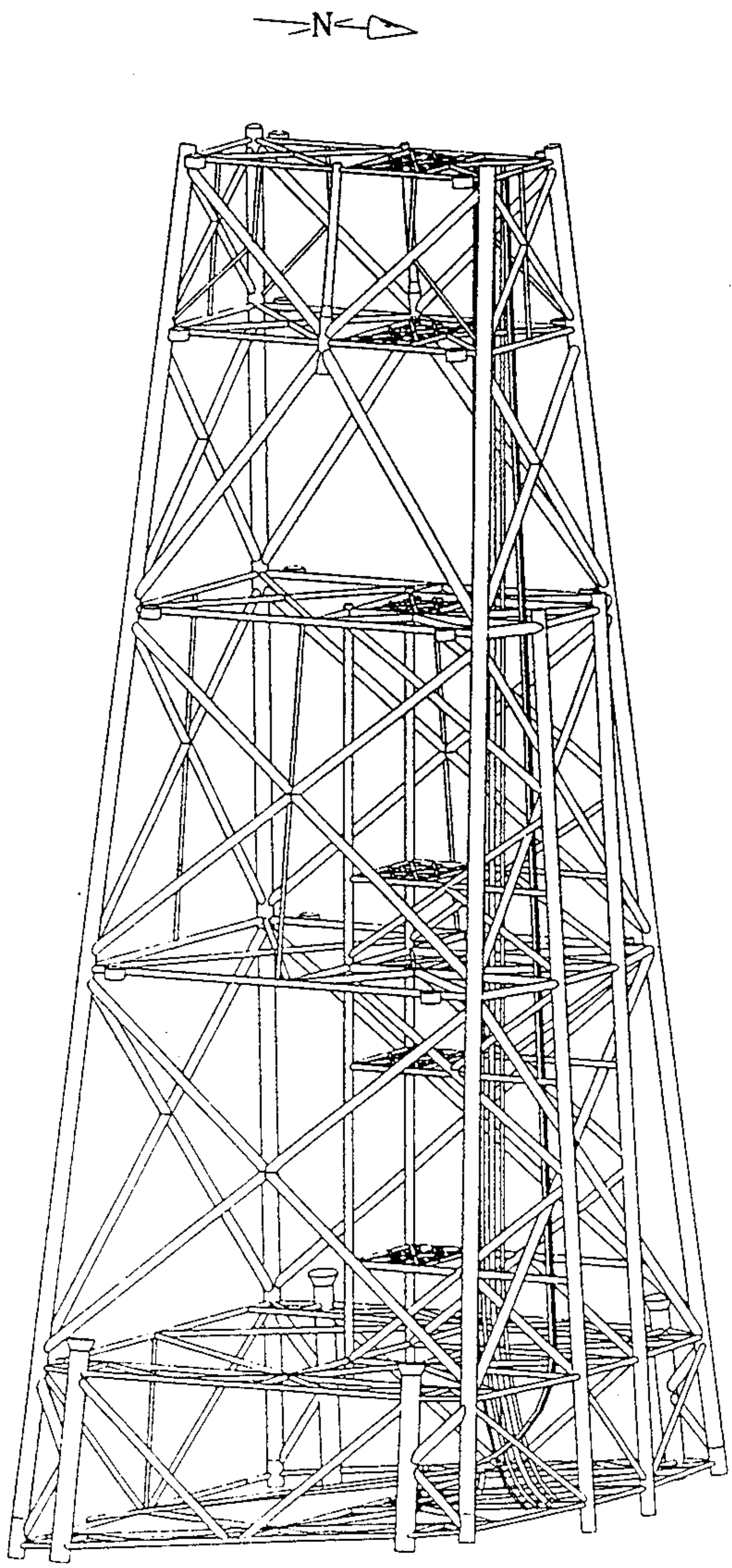
Platform MC397

By RKA Date / / Checked by Sheet No.

Project ANDREW JIP Job No. 295

Subject PLATFORM MC-397





PLATFORM MC 397

ALABASTER PLATFORM WAVE AND CURRENT FORCE RELATIONSHIPS
EXXON PRODUCTION RESEARCH COMPANY
JUNE 1993

COEFFICIENTS FOR THE FUNCTION: F(Shear or OTM) = C1 * (C2 * Hmax + C3 * Vcur)^C4

WAVE DIRECTION	COEFFICIENT	SHEAR	OTM
EAST	C1	1.000	1.00
	C2	0.645	24.52
	C3	3.644	117.67
	C4	2.141	1.91
	WIND (%)	8.8	12.2
	INERTIA (%)	17.5	18.4
SOUTHEAST	C1	1.000	1.00
	C2	0.582	18.91
	C3	3.521	97.95
	C4	2.175	1.97
	WIND (%)	11.8	16.7
	INERTIA (%)	9.1	9.4
SOUTH	C1	1.000	1.00
	C2	0.551	16.06
	C3	3.089	77.78
	C4	2.197	2.00
	WIND (%)	7.9	11.5
	INERTIA (%)	19.9	20.6

ALABASTER (MC-397)Platform: EXXON							
Water Depth =		**468'**					
WAVE and CURRENT Force Relationship							
Exxon's					**Equivalent Coefficients for**		
					API_RP_20th		
	BS	**OTM**				**BS**	**OTM**
East: BroadSide Direction							
C1 =	1	1			**C1=**	0.391	450.800
C2=	0.645	24.52					
C3=	3.644	117.67			**C2=**	**5.650**	**4.799**
C4=	2.141	1.91			**C3=**	**2.141**	**1.910**
Wind %ge	8.8	12.2			**C1w=**	0.42550	505.79781
Inertia %ge	17.5	18.4			**C1i =**	0.45952	533.74742
Combined Effect					**C1 =**	**0.49996**	**598.86461**
SouthEast: Diagonal Direction							
C1 =	1	1			**C1=**	0.308	327.403
C2=	0.582	18.91					
C3=	3.521	97.95			**C2=**	**6.050**	**5.180**
C4=	2.175	1.97			**C3=**	**2.175**	**1.970**
Wind %ge	11.8	16.7			**C1w=**	0.34447	382.07900
Inertia %ge	9.1	9.4			**C1i =**	0.33615	358.17860
Combined Effect					**C1 =**	**0.37581**	**417.99443**
South: EndOn Direction							
C1 =	1	1			**C1=**	0.270	257.924
C2=	0.551	16.06					
C3=	3.089	77.78			**C2=**	**5.606**	**4.843**
C4=	2.197	2			**C3=**	**2.197**	**2.000**
Wind %ge	7.9	11.5			**C1w=**	0.29129	287.58481
Inertia %ge	19.9	20.6			**C1i =**	0.32369	311.05586
Combined Effect					**C1 =**	**0.34926**	**346.82729**

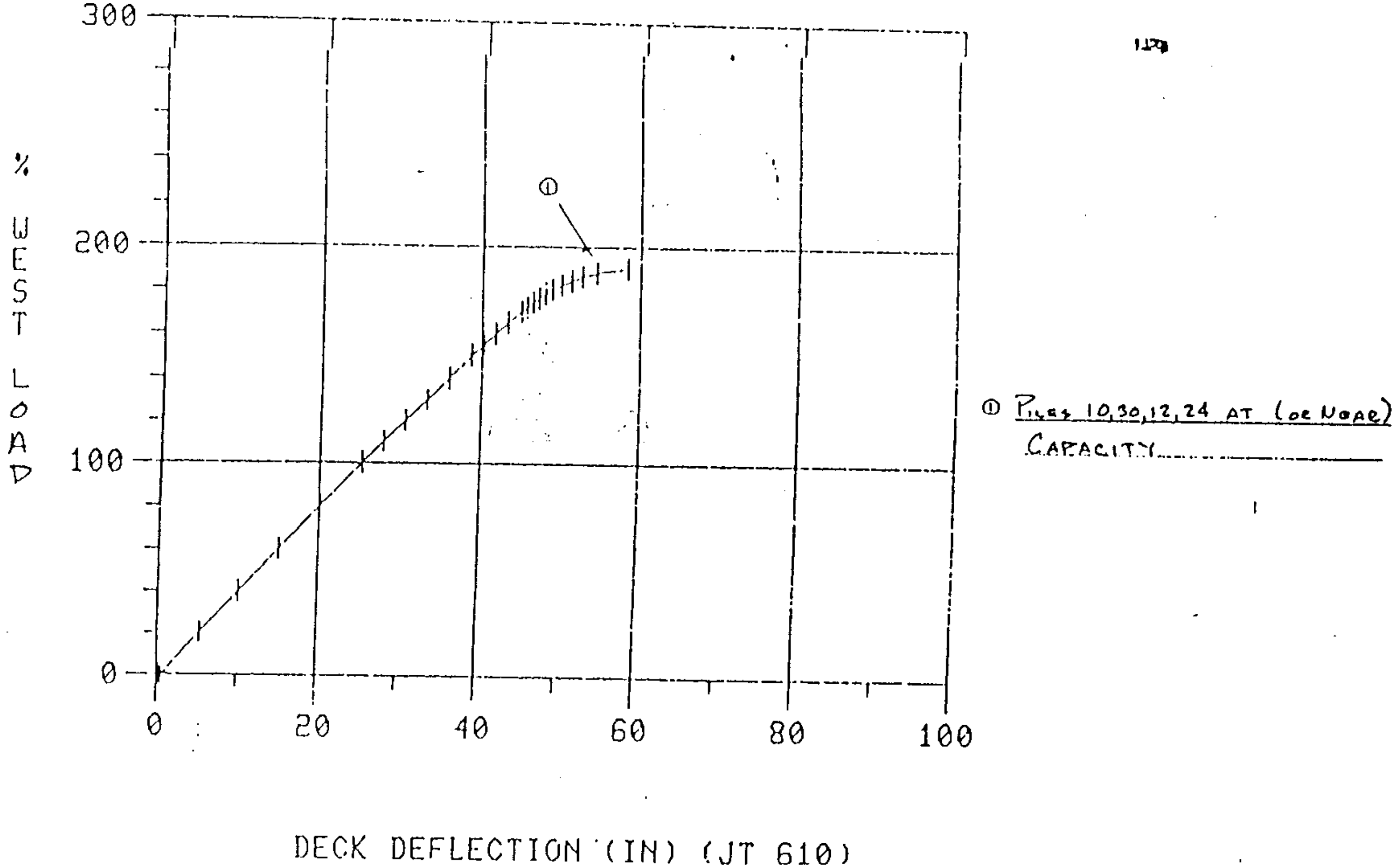
PLATFORM MC397

This Platform Survived During Hurricane Andrew Loads

Water Depth = 468 ft.

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.709 * Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossin Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Broadside Direction:			(262.5 deg. to 307.5 deg.)										13718.00
3	260.6	26.61	45.462	12.885	9.53	0.49996	5.65	2.141	0.61	48.884	2067.46		
4	264.0	29.96	51.187	13.094	9.69	0.49996	5.65	2.141	0.81	55.736	2737.86		
5	273.4	33.62	57.437	13.314	9.85	0.49996	5.65	2.141	1.05	63.346	3600.91		
6	290.4	37.59	64.234	13.794	10.21	0.49996	5.65	2.141	1.25	71.322	4641.81		
7	273.4	38.57	65.901	13.641	10.09	0.49996	5.65	2.141	1.33	73.415	4938.33		
8	290.4	36.41	62.209	13.278	9.83	0.49996	5.65	2.141	1.27	69.402	4378.39		
Diagonal Direction:			(307.5 deg. to 352.5 deg.)										11566.00
9	313.0	33.59	57.388	12.458	9.22	0.37581	6.05	2.175	1.03	63.596	3143.60		
10	316.3	31.38	53.618	11.924	8.82	0.37581	6.05	2.175	0.70	57.826	2556.17		
11	316.1	29.72	50.780	11.779	8.72	0.37581	6.05	2.175	0.39	53.163	2128.99		
12	315.8	27.83	47.550	11.386	8.43	0.37581	6.05	2.175	0.14	48.391	1735.09		
13	318.0	25.72	43.941	10.820	8.01	0.37581	6.05	2.175	0.00	43.941	1406.75		
14	320.9	24.17	41.293	10.683	7.91	0.37581	6.05	2.175	0.00	41.293	1228.82		
15	321.9	23.05	39.387	10.618	7.86	0.37581	6.05	2.175	0.00	39.387	1108.80		
16	322.8	21.88	37.383	10.520	7.78	0.37581	6.05	2.175	0.00	37.383	989.75		

% WEST(EAST) LOAD VS DECK DEFLECTION
ALABASTER-6/15/89 - WEST



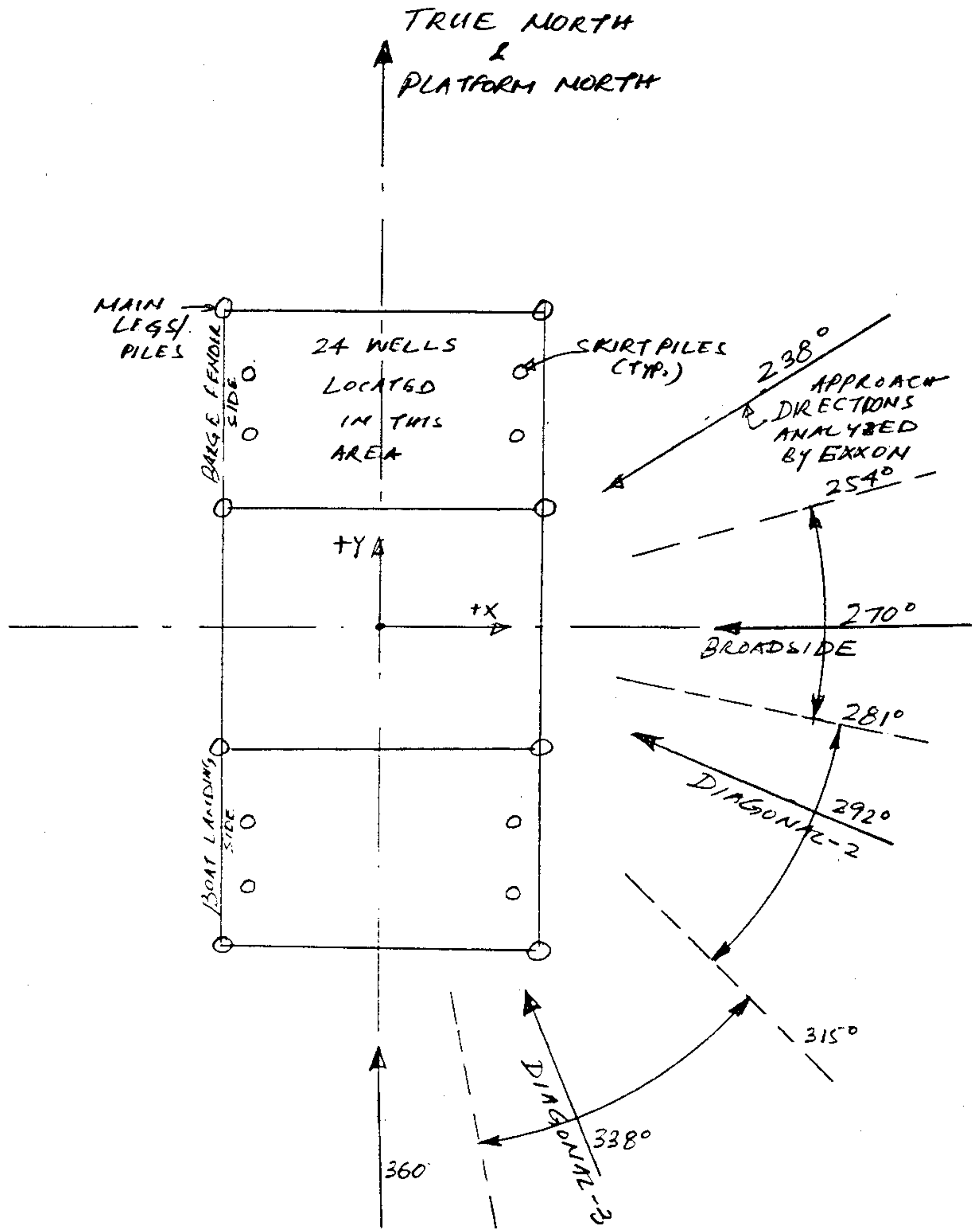
Platform MC311

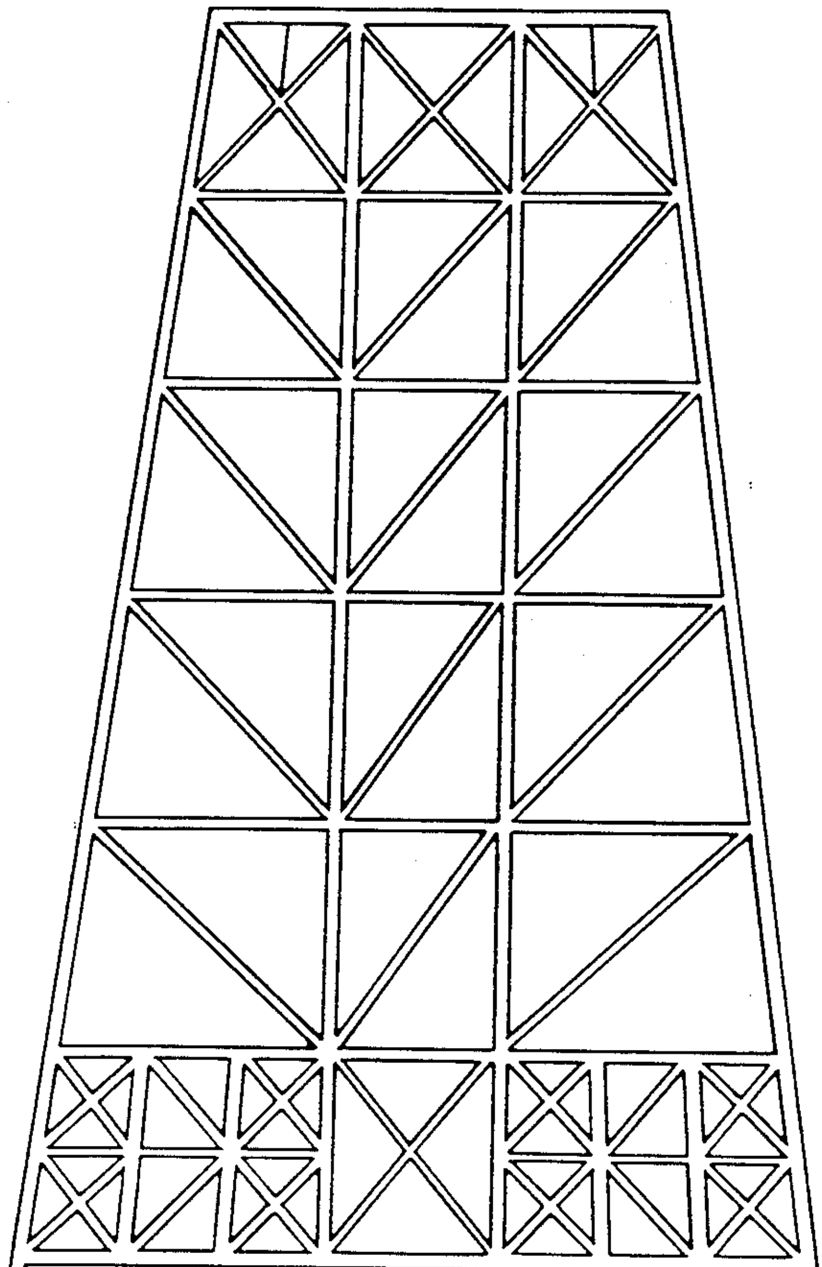
PMB ENGINEERING

By: RKA | Date: / / | Checked by: | Sheet No.:

Project: ANDREW JP | Job No.: 295

Subject: PLATFORM MC311





MC311 COLUMN ROW 1 (WEST) ELEVATION

MC-311 PLATFORM WAVE AND CURRENT FORCE RELATIONSHIPS
EXXON PRODUCTION RESEARCH COMPANY
JUNE 1993

COEFFICIENTS FOR THE FUNCTION: F(Shear or OTM) = C1 * (C2 * Hmax + C3 * Vcur)^C4

WAVE DIRECTION	COEFFICIENT	SHEAR	OTM
SOUTH	C1	1.000	1.00
	C2	0.438	6.51
	C3	2.368	28.05
	C4	2.474	2.31
	WIND (%)	3.3	5.5
SOUTH-SOUTHEAST	C1	1.000	1.00
	C2	0.441	6.36
	C3	2.334	26.68
	C4	2.470	2.32
	WIND (%)	6.0	9.7
EAST-SOUTHEAST	C1	1.000	1.00
	C2	0.531	8.94
	C3	2.882	38.12
	C4	2.384	2.23
	WIND (%)	7.4	11.6
EAST	C1	1.000	1.00
	C2	0.562	9.95
	C3	2.862	39.93
	C4	2.360	2.20
	WIND (%)	6.6	10.2
EAST-NORTHEAST	C1	1.000	1.00
	C2	0.532	8.64
	C3	2.728	34.82
	C4	2.381	2.24
	WIND (%)	7.6	12.0

MC-311 Platform: EXXON							
Water Depth =		343	ft.				
WAVE and CURRENT Force Relationship							
					Equivalent Coefficients for		
Exxon's					API_RP_20th		
	BS	OTM				BS	OTM
South: EndOn Direction				349 deg to 382.5 deg			
C1 =	1	1			C1=	0.12972	75.74816
C2=	0.438	6.51					
C3=	2.368	28.05			C2=	5.40639	4.30876
C4=	2.474	2.31			C3=	2.47400	2.31000
Wind %ge	3.3	5.5			C1=	0.13400	79.91431
South-SouthEast: Diagonal_3 Direction				315 deg to 349 deg			
C1 =	1	1			C1=	0.13236	73.11736
C2=	0.441	6.36					
C3=	2.334	26.68			C2=	5.29252	4.19497
C4=	2.47	2.32			C3=	2.47000	2.32000
Wind %ge	6	9.7			C1=	0.14030	80.20975
East-SouthEast: Diagonal_2 Direction				281 deg to 315 deg			
C1 =	1	1			C1=	0.22112	132.27649
C2=	0.531	8.94					
C3=	2.882	38.12			C2=	5.42750	4.26398
C4=	2.384	2.23			C3=	2.38400	2.23000
Wind %ge	7.4	11.6			C1=	0.23748	147.62056
East: Broadside Direction				254 deg to 281 deg			
C1 =	1	1			C1=	0.25667	156.75117
C2=	0.562	9.95					
C3=	2.862	39.93			C2=	5.09253	4.01307
C4=	2.36	2.2			C3=	2.36000	2.20000
Wind %ge	6.6	10.2			C1=	0.27361	172.73978
East_NorthEast: Diagonal_1 Direction				225 deg to 254 deg			
C1 =	1	1			C1=	0.22253	125.25376
C2=	0.532	8.64					
C3=	2.728	34.82			C2=	5.12782	4.03009
C4=	2.381	2.24			C3=	2.38100	2.24000
Wind %ge	7.6	12			C1=	0.23945	140.28421

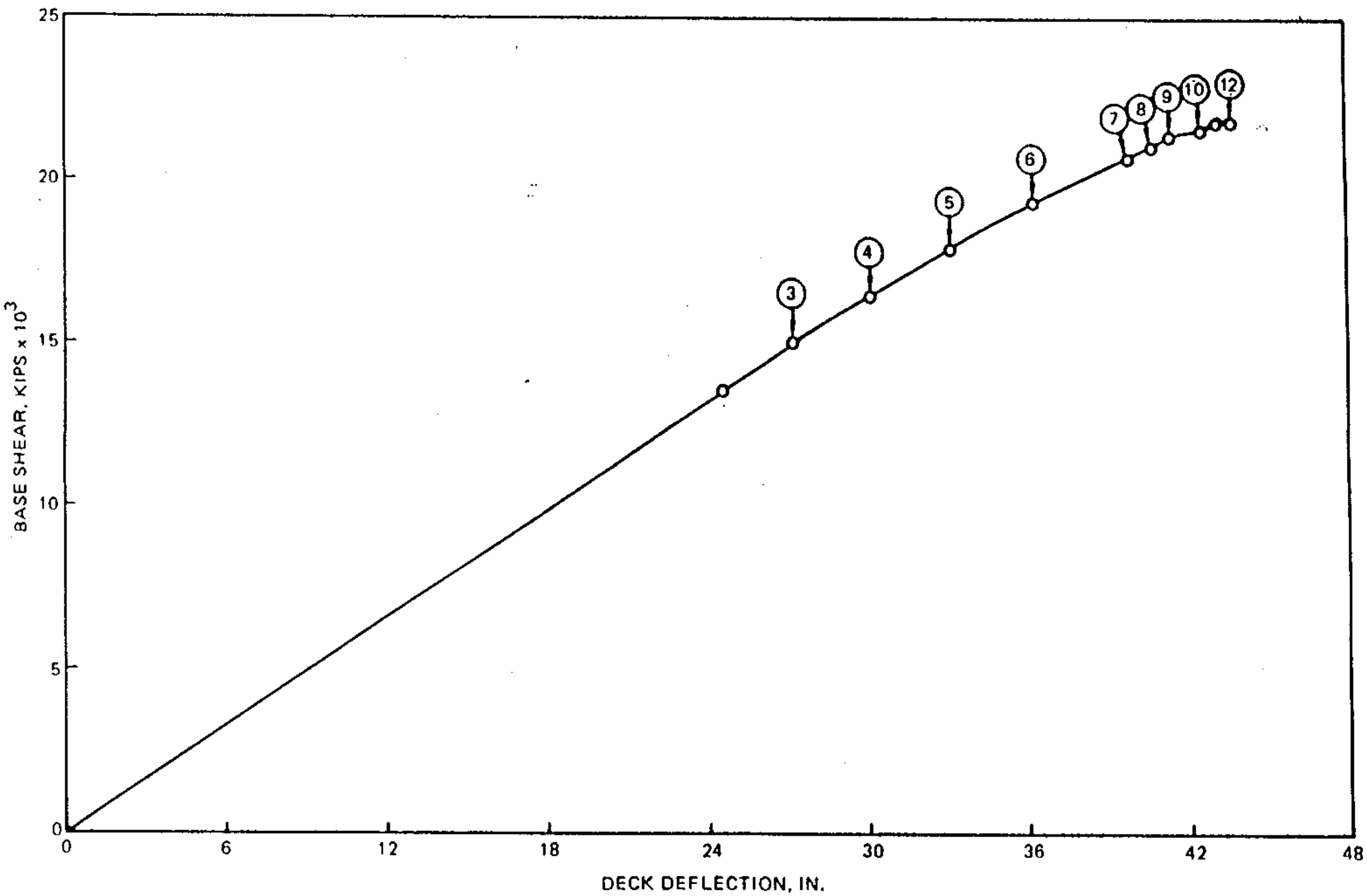
PLATFORM MC311

Water Depth = 343 ft.

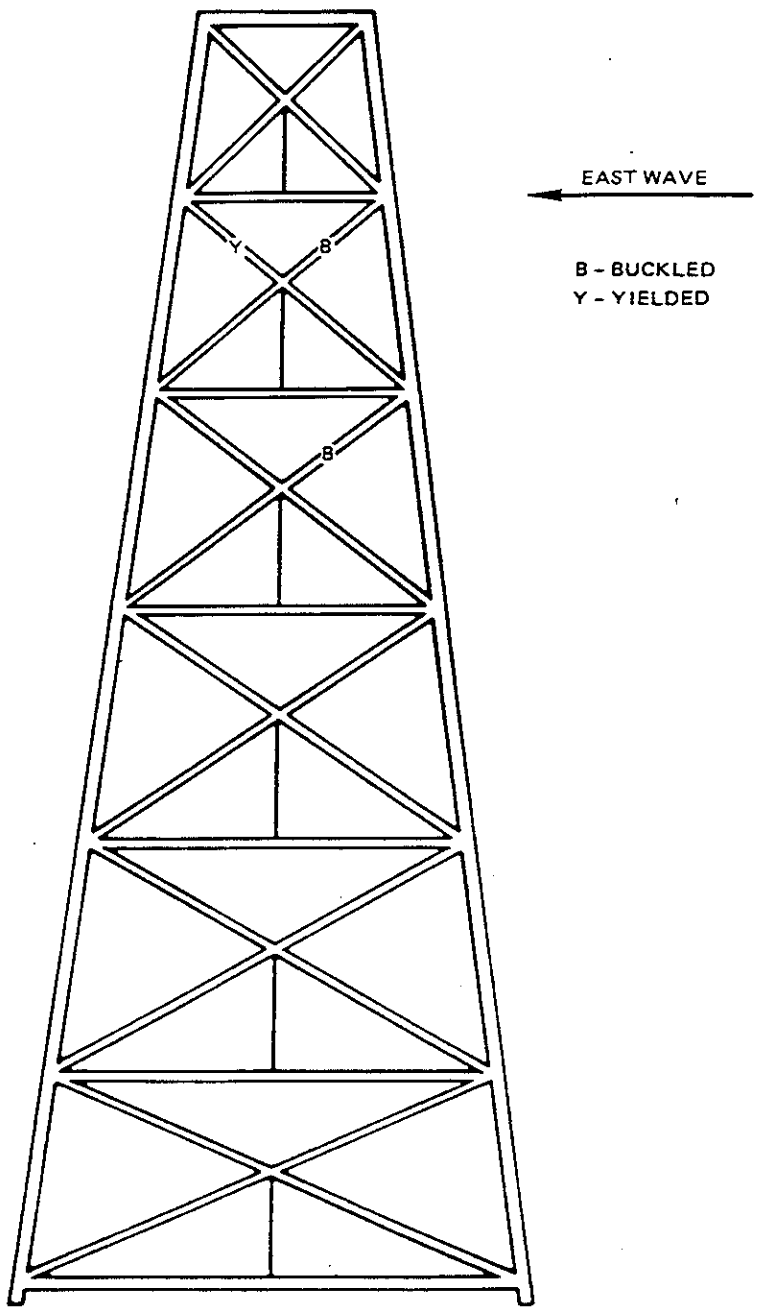
This Platform Survived During Hurricane Andrew Loads

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.725 * Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Broadside Direction:			(254 deg. to 281 deg.)										20700.00
1	262.0	20.18	34.805	11.609	8.59	0.27361	5.09253	2.36	0.50	37.373	1407.24		
2	264.0	23.07	39.792	11.992	8.87	0.27361	5.09253	2.36	0.64	43.028	1962.37		
3	266.1	25.84	44.571	12.445	9.21	0.27361	5.09253	2.36	0.82	48.766	2636.90		
4	268.0	28.46	49.092	12.653	9.36	0.27361	5.09253	2.36	1.06	54.503	3428.32		
5	271.6	31.17	53.769	12.825	9.49	0.27361	5.09253	2.36	1.34	60.601	4403.28		
6	278.5	34.13	58.877	13.304	9.85	0.27361	5.09253	2.36	1.62	67.131	5606.16		
Diagonal-2 Direction:			(281 deg. to 315 deg.)										17900.00
7	288.3	35.95	62.006	13.513	10.00	0.23748	5.4275	2.384	1.86	72.100	6381.89		
8	298.5	35.50	61.231	13.150	9.73	0.23748	5.4275	2.384	1.95	71.820	6322.98		
9	307.5	33.17	57.224	12.621	9.34	0.23748	5.4275	2.384	1.74	66.685	5298.02		
10	311.8	31.03	53.534	12.090	8.95	0.23748	5.4275	2.384	1.38	61.005	4284.90		
11	314.3	29.10	50.203	11.783	8.72	0.23748	5.4275	2.384	0.99	55.575	3431.05		
Diagonal-3 Direction:			(315 deg. to 349 deg.)										17400.00
12	316.0	27.29	47.080	11.340	8.39	0.1403	5.29252	2.47	0.65	50.522	2262.81		
13	318.2	25.48	43.950	10.940	8.10	0.1403	5.29252	2.47	0.40	46.065	1801.27		
14	321.3	23.93	41.279	10.719	7.93	0.1403	5.29252	2.47	0.12	41.898	1425.18		
15	322.7	22.82	39.367	10.650	7.88	0.1403	5.29252	2.47	0.00	39.367	1221.91		
16	323.7	21.73	37.480	10.580	7.83	0.1403	5.29252	2.47	0.00	37.480	1082.28		
17	324.7	20.55	35.448	10.461	7.74	0.1403	5.29252	2.47	0.00	35.448	943.07		
18	326.5	19.37	33.413	10.209	7.55	0.1403	5.29252	2.47	0.00	33.413	814.95		

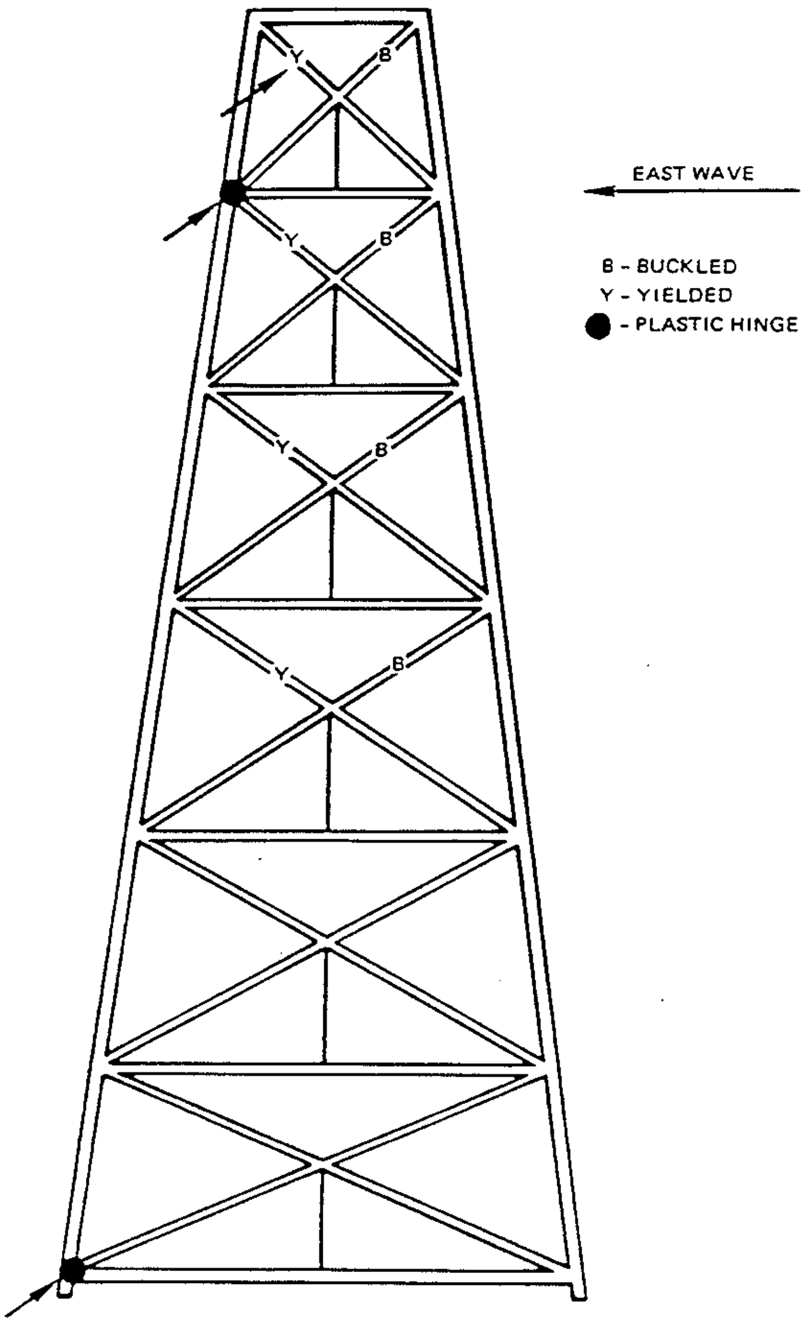
NOTE: 1. THE SEASTATE DATA AND PF RESULTS ARE SAME AS IN THE DRAFT REPORT OF AUGUST, 1993.
REVISED DATA WITH NEW INTERPOLATION IS EXPECTED TO BE SIMILAR.
2. THIS PLATFORM DID NOT HAVE MUCH EFFECT ON CALIBRATION [illegible]



LATERAL DECK DEFLECTION-BASE SHEAR FOR EAST WAVE PUSHOVER



EAST WAVE INTRA ANALYSIS - LOAD STEP 3



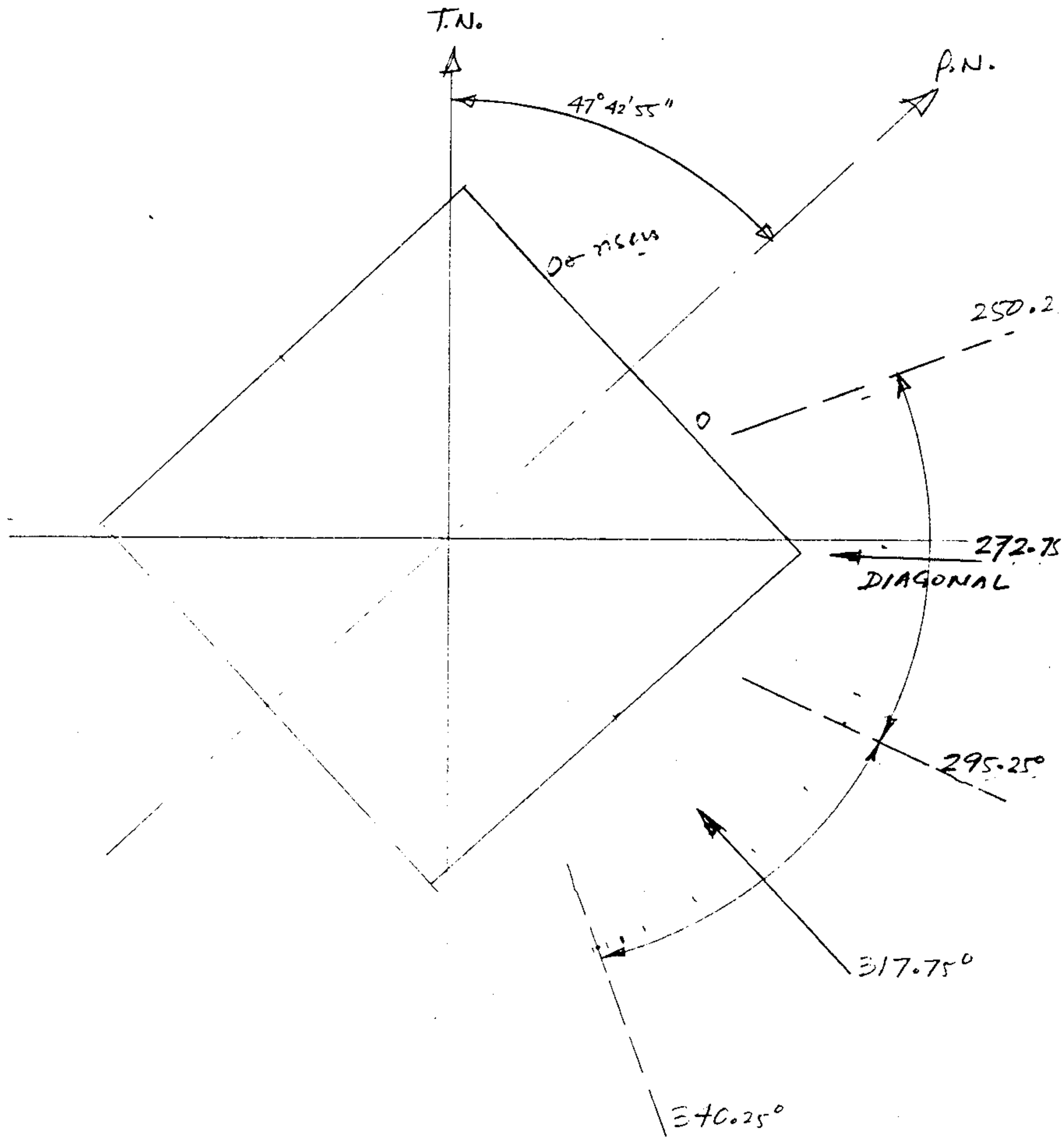
EAST WAVE INTRA ANALYSIS - LOAD STEP 12

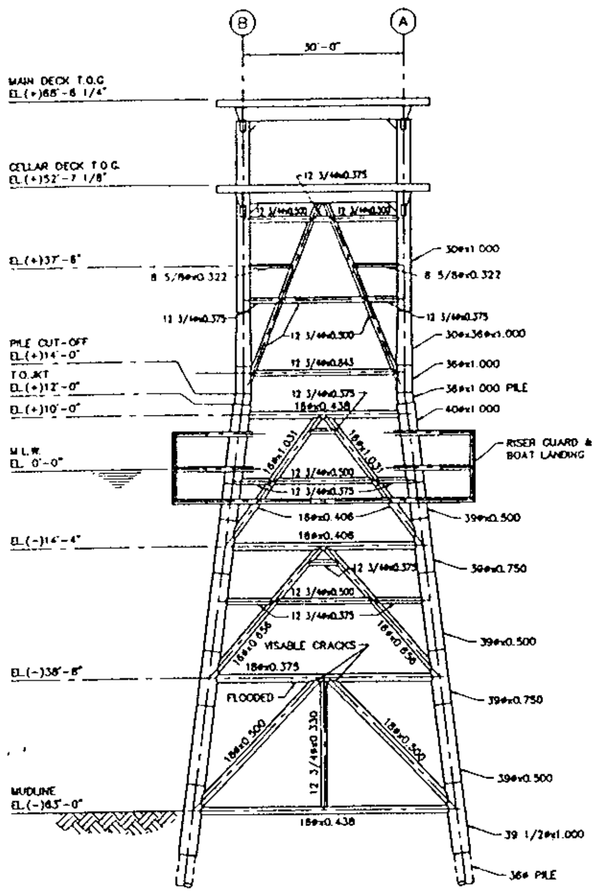
Platform T23

By RKA Date 06/07/93 Checked by Sheet No.

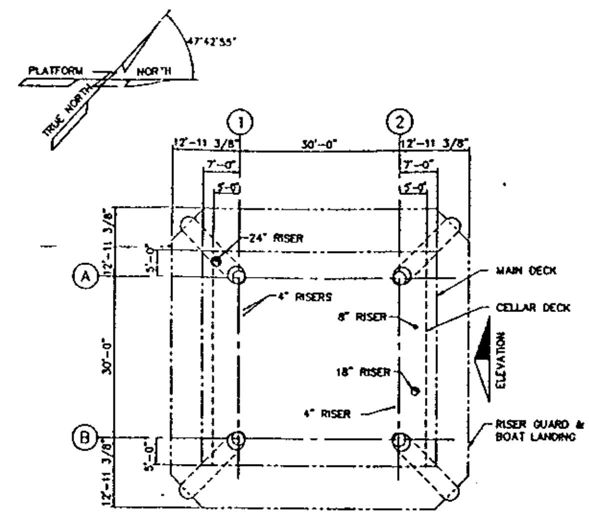
Project ANDREW JIP Job No.

Subject PLATFORM ST 52 T-2B





NORTH ELEVATION



PLAN

NOTES

RELEASED FOR	DATE	BY	APPD
Information			
Preliminary			
Bidding			
Client Approval			

CBS ENGINEERING, INC.
Houston, Texas

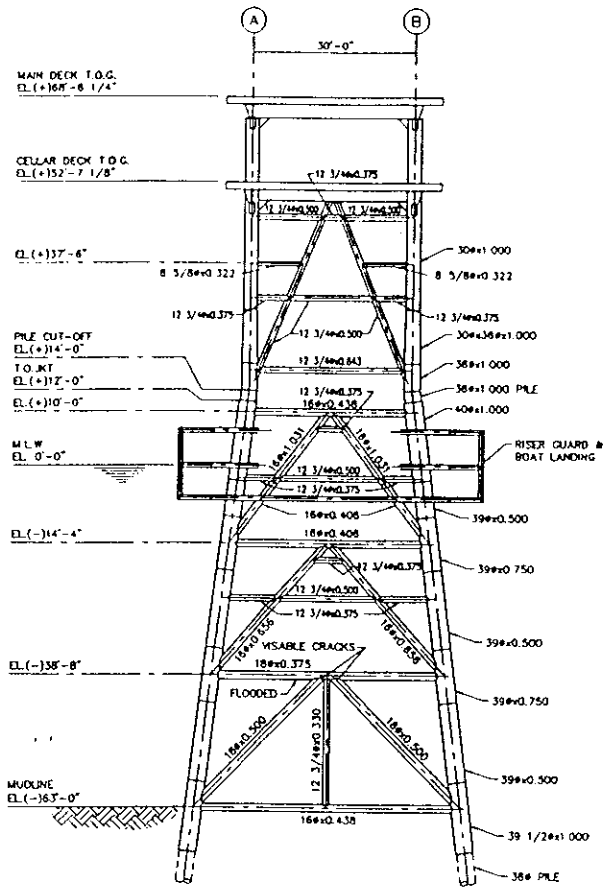
scale 3/32"=1'-0"
drawn by M. POERNER date SEP. '92

TRUNKLINE GAS COMPANY

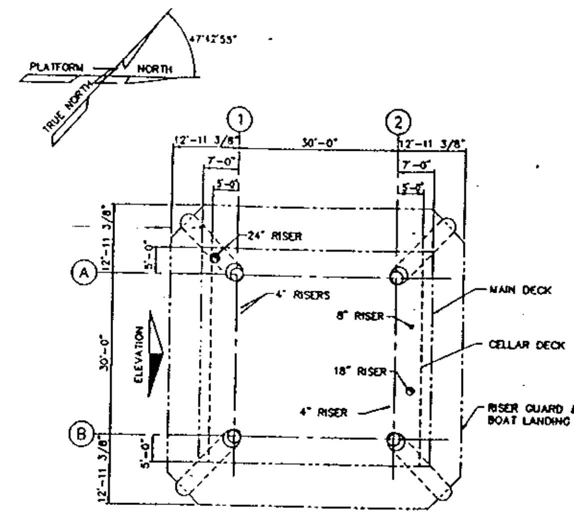
T-23 GAS GATHERING PLATFORM

SOUTH TIMBALIER BLK 52 63' W.D.

NORTH ELEVATION



SOUTH ELEVATION



PLAN

RELEASED FOR	DATE	BY	APPD
Information			
Preliminary			
Bidding			
Client Approval			

CBS ENGINEERING, INC.
Houston, Texas

Scale: 3/32"=1'-0"
Drawn by: M. POERNER Date: SEP. '92

TRUNKLINE GAS COMPANY

T-23 GAS GATHERING PLATFORM
SOUTH TIMBALIER BLK 52 63' W.D.
SOUTH ELEVATION

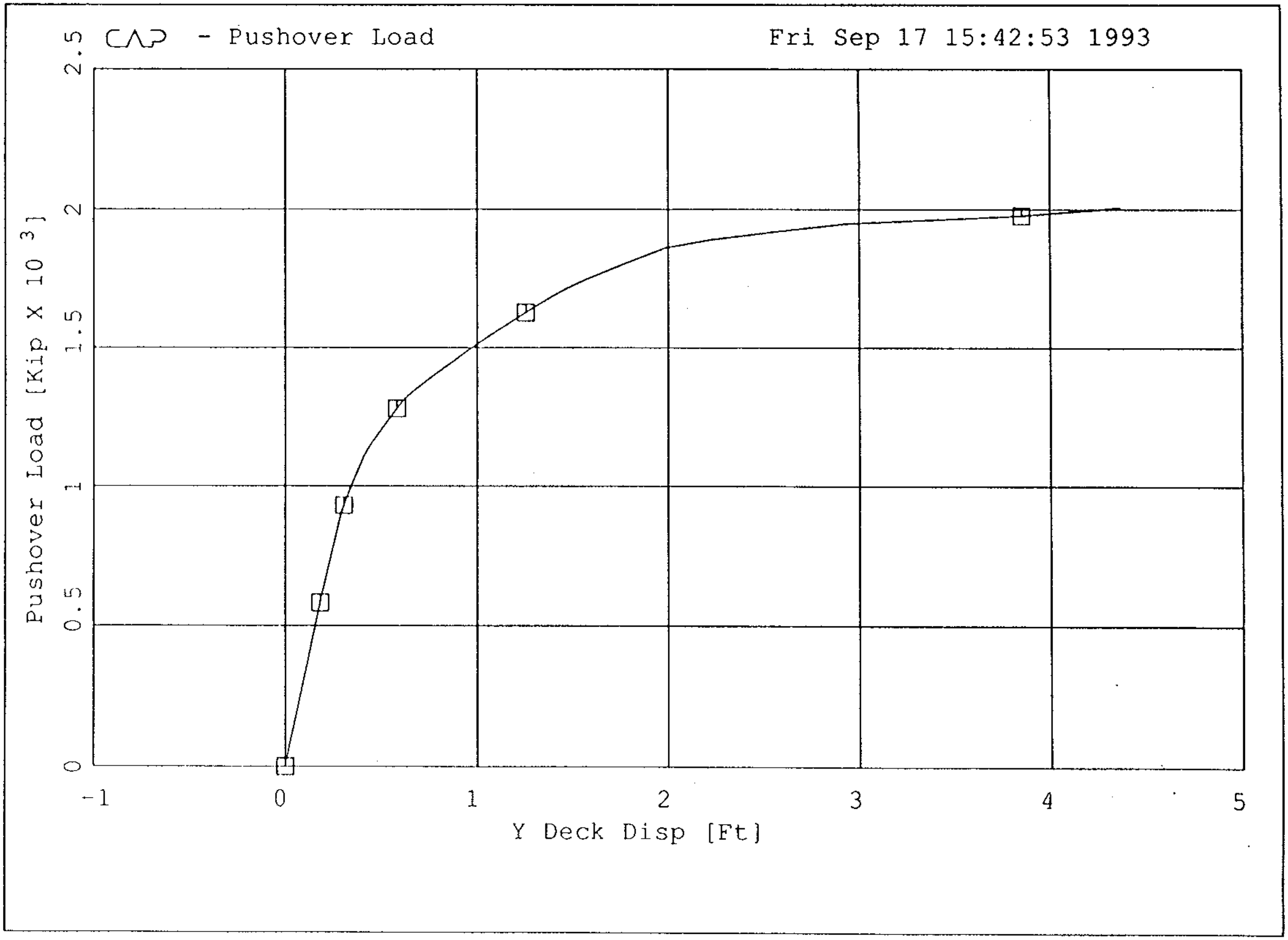
PLATFORM ST52 (T23)

This Platform Damaged Against Hurricane Andrew Loads

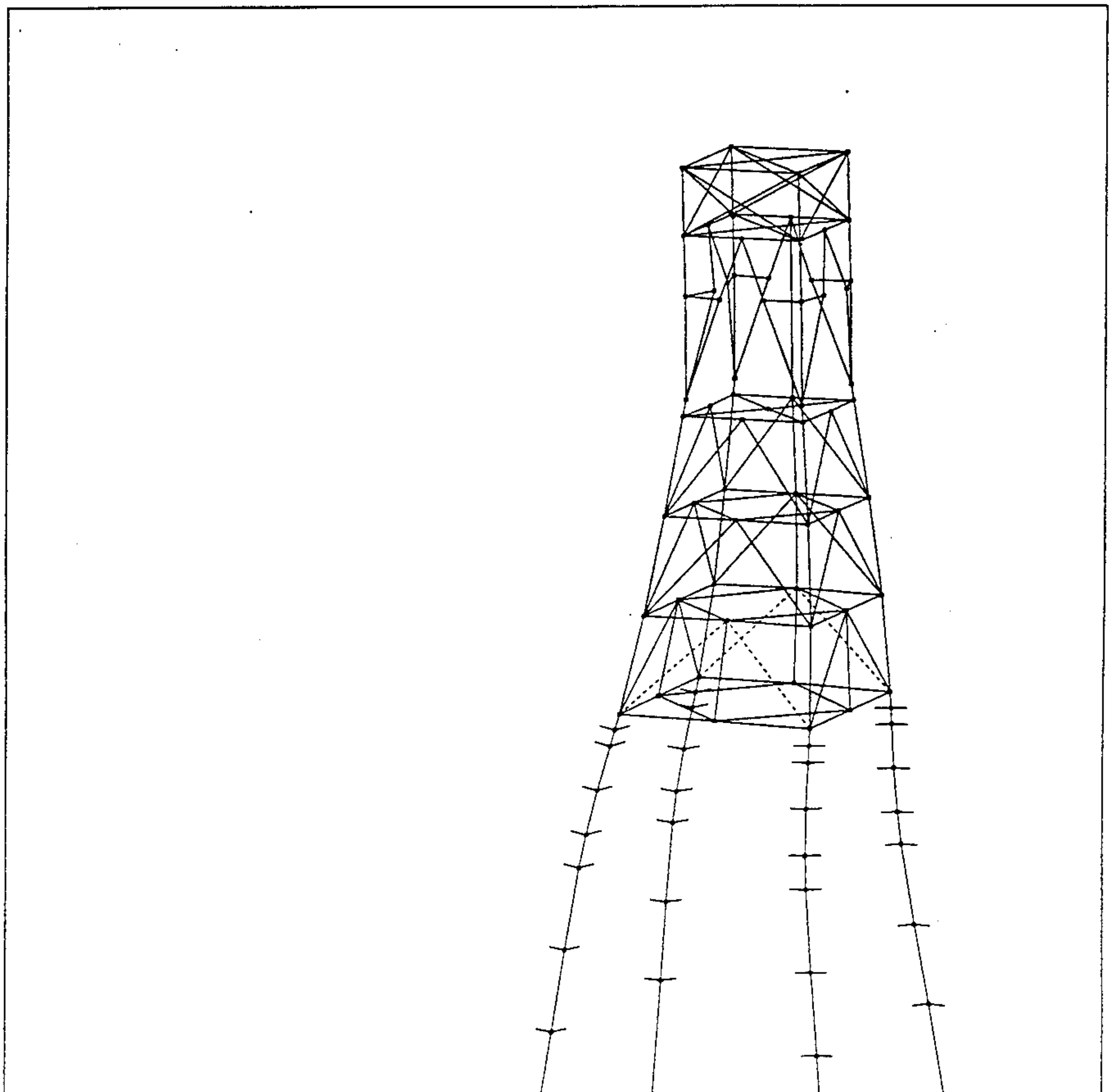
Water Depth = 63.00 ft.

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.700 *Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossin Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	U Platform (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Diagonal_1 Direction			(250.25 deg. to 284 deg.)											
1	255.07	18.31	31.12	10.01	7.40	0.272244	2.34517	2.03881	2.27	1.93	35.65	397.42		
2	256.30	19.93	33.89	10.40	7.70	0.272244	2.34517	2.03881	2.77	2.35	39.40	487.45		
3	262.48	22.40	38.08	10.74	7.95	0.272244	2.34517	2.03881	3.34	2.84	44.75	631.91		
4	271.76	27.44	46.64	14.50	10.73	0.272244	2.34517	2.03881	3.99	3.39	54.60	947.84		
Diagonal_2 Direction			(284 deg. to 306.5 deg.)											2006.00
5	286.37	29.59	50.30	14.61	10.81	0.272244	2.34517	2.03881	4.13	3.51	58.53	1092.15		
6	296.73	28.51	48.46	14.00	10.36	0.272244	2.34517	2.03881	3.48	2.96	55.41	976.80		
7	304.87	27.64	46.99	13.56	10.03	0.272244	2.34517	2.03881	2.32	1.98	51.62	845.48		
Orthogonal Direction			(306.5 deg. to 340.25 deg.)											
8	312.21	25.40	43.18	13.19	9.76	0.296518	2.24518	2.02095	1.32	1.06	45.55	666.35		
9	318.54	24.10	40.97	12.11	8.96	0.296518	2.24518	2.02095	0.70	0.56	42.23	571.84		
10	324.05	22.78	38.73	11.83	8.76	0.296518	2.24518	2.02095	0.15	0.12	39.00	486.97		
11	326.56	21.13	35.92	11.64	8.61	0.296518	2.24518	2.02095	0.00	0.00	35.92	412.41		
12	328.99	19.77	33.61	10.82	8.01	0.296518	2.24518	2.02095	0.00	0.00	33.61	360.59		
13	331.61	18.61	31.64	10.58	7.83	0.296518	2.24518	2.02095	0.00	0.00	31.64	319.11		
14	332.19	17.46	29.69	10.43	7.72	0.063951	1.79354	2.47105	0.00	0.00	29.69	278.44		
15	332.34	16.33	27.76	10.10	7.48	0.063951	1.79354	2.47105	0.00	0.00	27.76	235.88		
16	332.89	15.45	26.27	9.64	7.14	0.063951	1.79354	2.47105	0.00	0.00	26.27	205.73		

Project: T23 Model: diag2 Version: 2



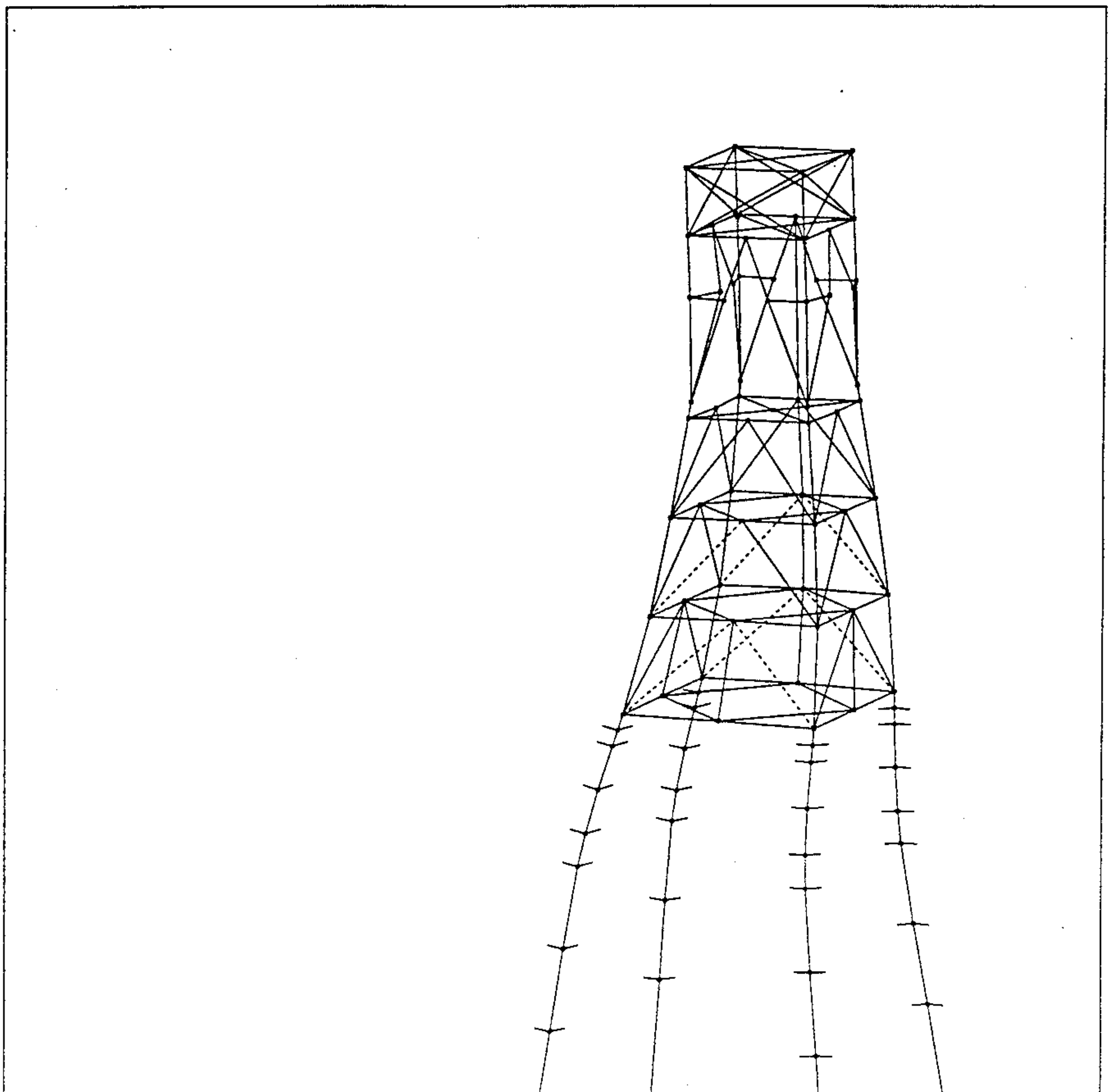
T23 Diag2 Dir.



CAP

T23 Diag-2 Dir. Events_LS27

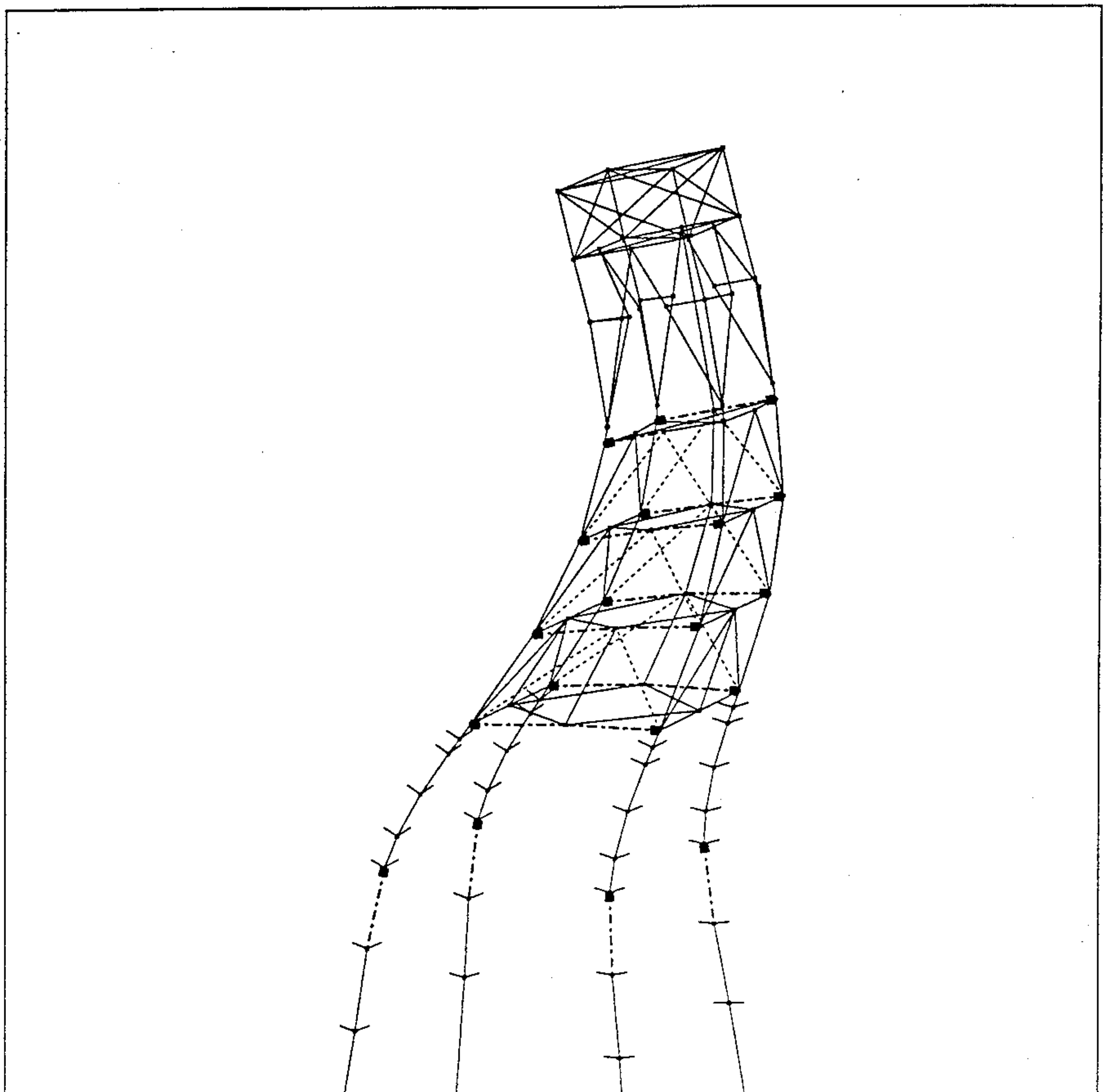
Project: T23 Model: diag2 Version: 2



CAP

T23 Diag-2 Dir. Events_LS31

Project: T23 Model: diag2 Version: 2



CAP

T23 Diag-2 Dir. Events_LS52

Project: T23 Model: diag2 Version: 2

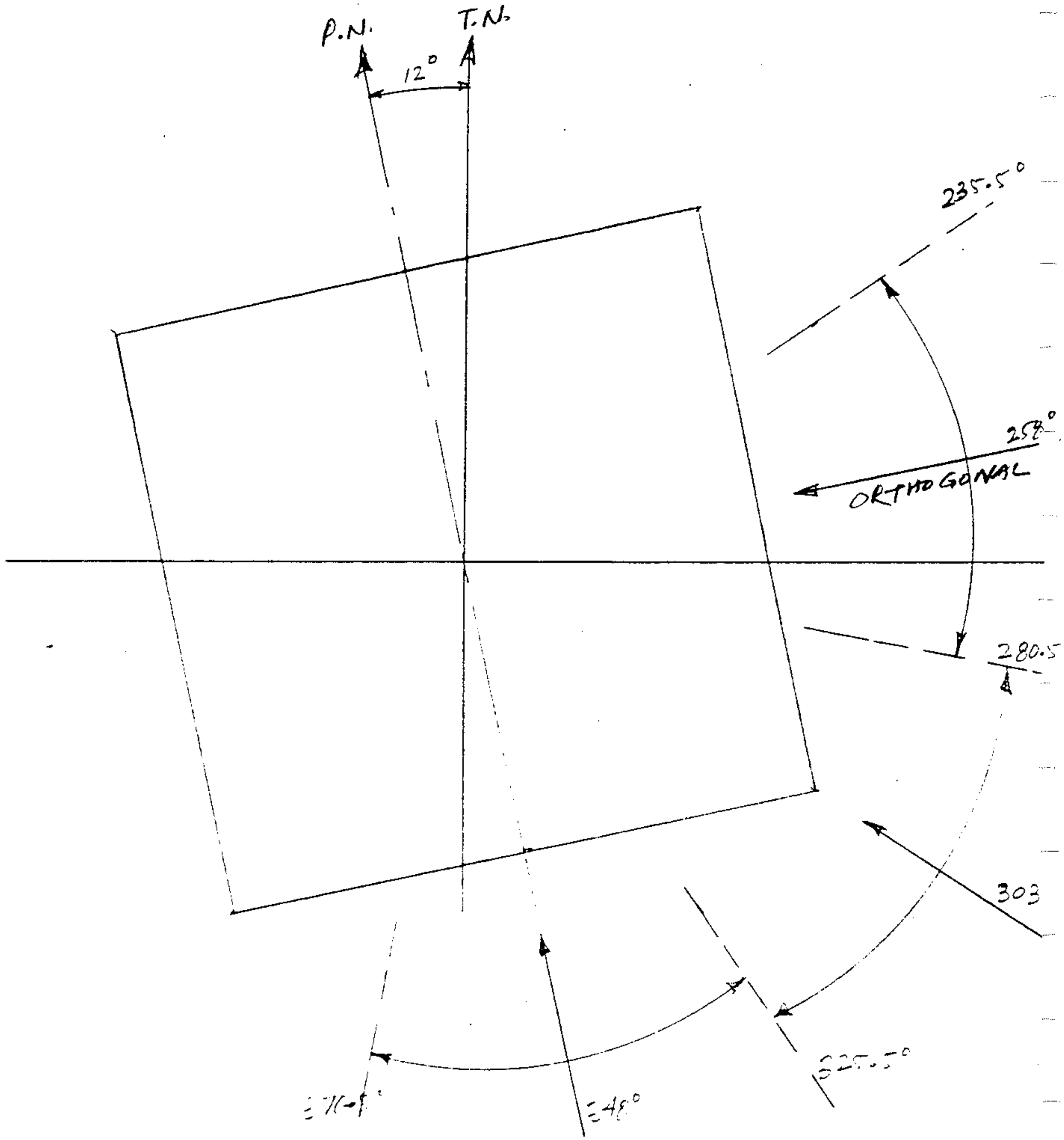
Platform T25

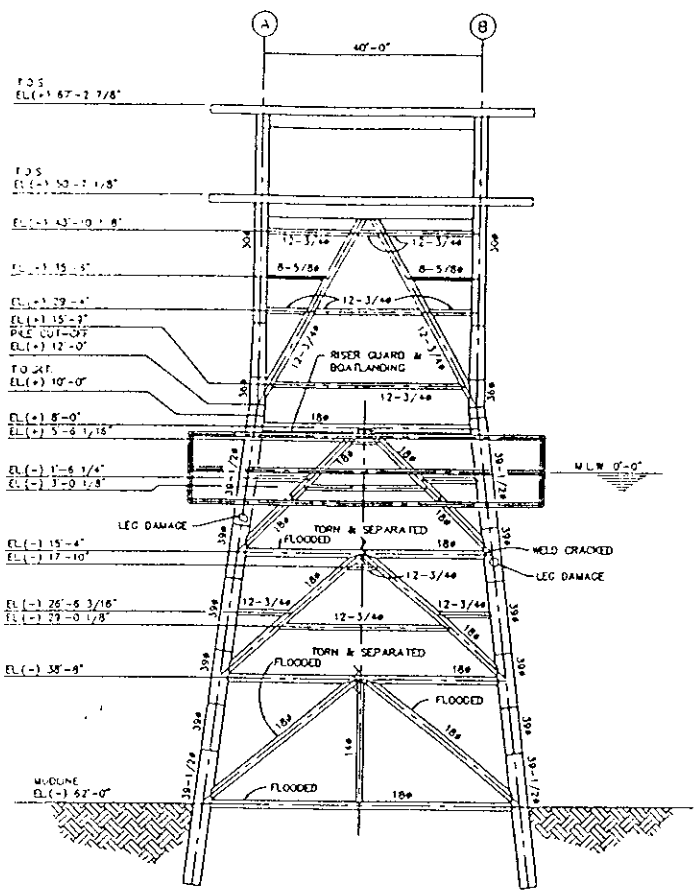
PMB ENGINEERING

By RKA Date 06/16/93 Checked by____ Sheet No.____

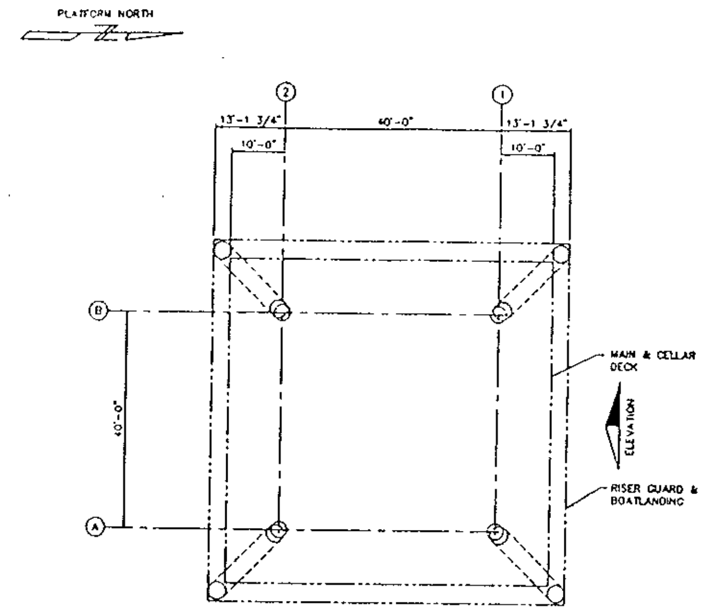
Project ANDREW JIP Job No. 295

Subject PLATFORM SS 139, T25





NORTH ELEVATION



PLAN

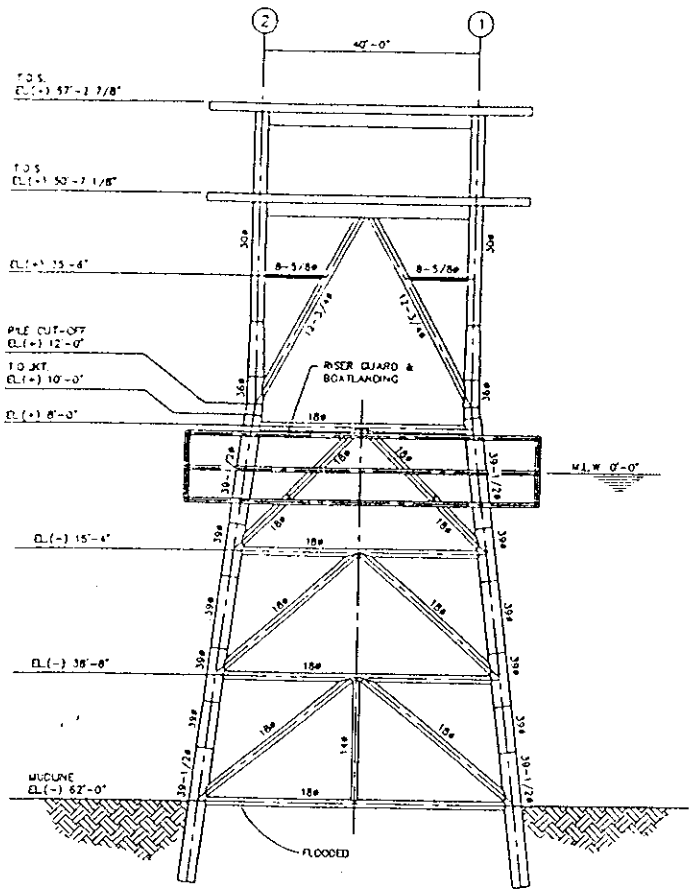
RELEASED FOR	DATE	BY	APPD
Information			
Preliminary			
Bidding			
Client Approval			

CBS ENGINEERING, INC.
Houston, Texas

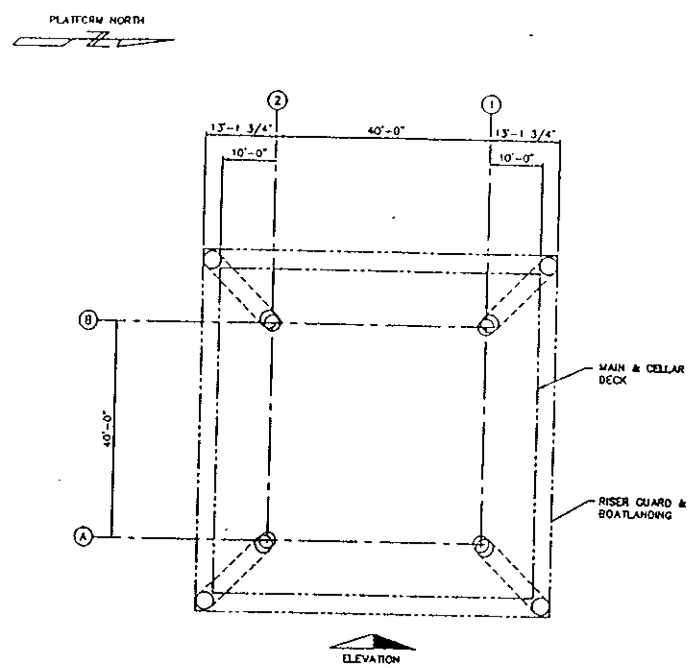
scale 3/32" = 1'-0"
KEITH DUAN — AUG. '93

TRUNKLINE GAS COMPANY

60' x 60' HUB PLATFORM
SHIP SHOAL BLK. 139 (T-23) 62' W.D.



EAST ELEVATION



PLAN

RELEASED FOR	DATE	BY	APPD
Information			
Preliminary			
Bidding			
ent. Apprv			

CBS ENGINEERING, INC.
Houston, Texas

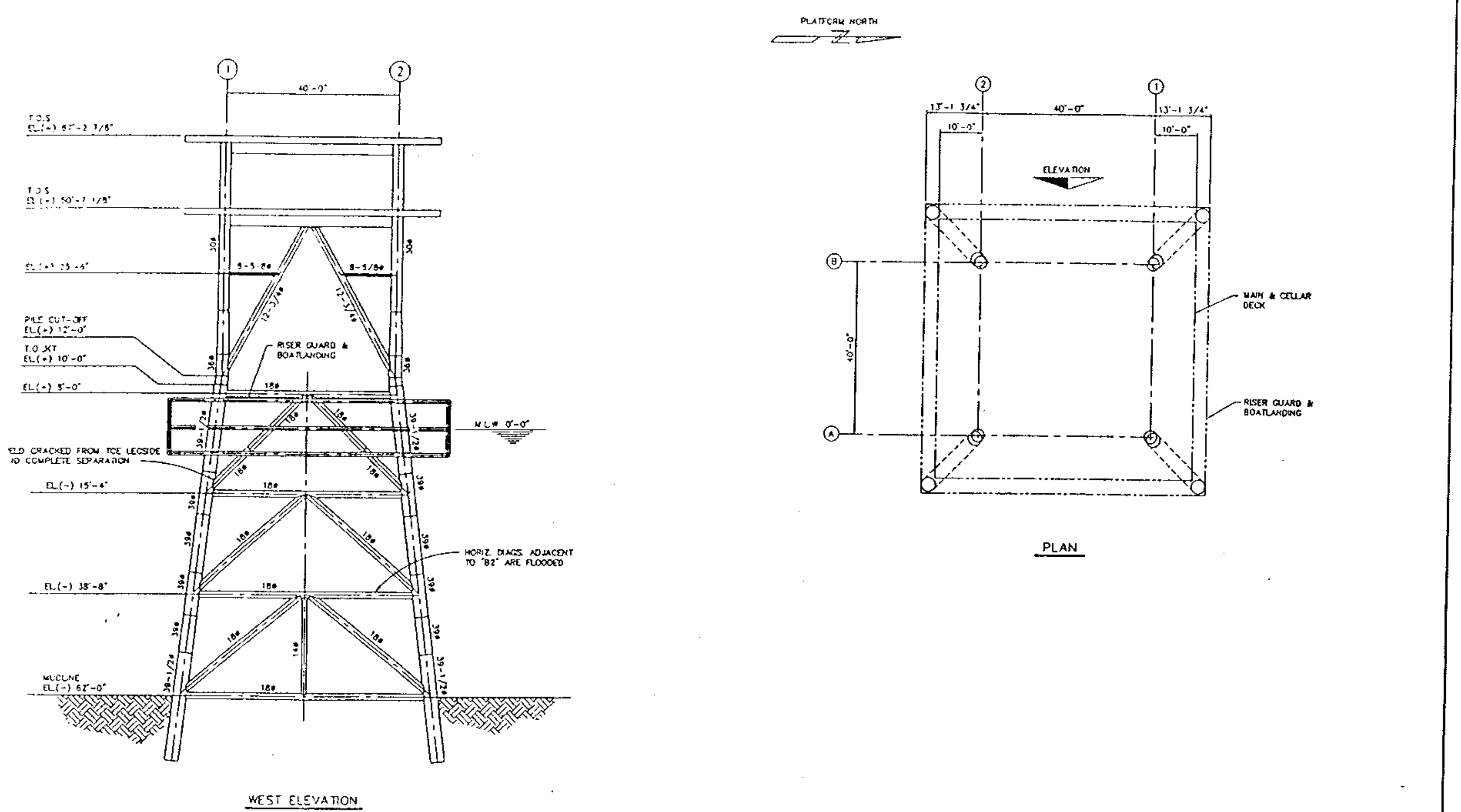
A. PEYREFITTE — scale 3/32"=1'-0" — date AUG '99

TRUNKLINE GAS COMPANY

T-25 60' x 60' HUB PLATFORM

SHIP SHOAL BLK. 139 — 62' W.D.

ELEVATION



RELEASED FOR	DATE	BY	APPD
Information			
Preliminary			
Bidding			
Client Approval			

CBS ENGINEERING, INC.
Houston, Texas

Drawn by A. PEYREFITTE — Scale 3/32"=1'-0" — Date AUG. '92

TRUNKLINE GAS COMPANY

60' x 60' HUB PLATFORM

SHIP SHOAL BLK. 139 — 62' W.D.

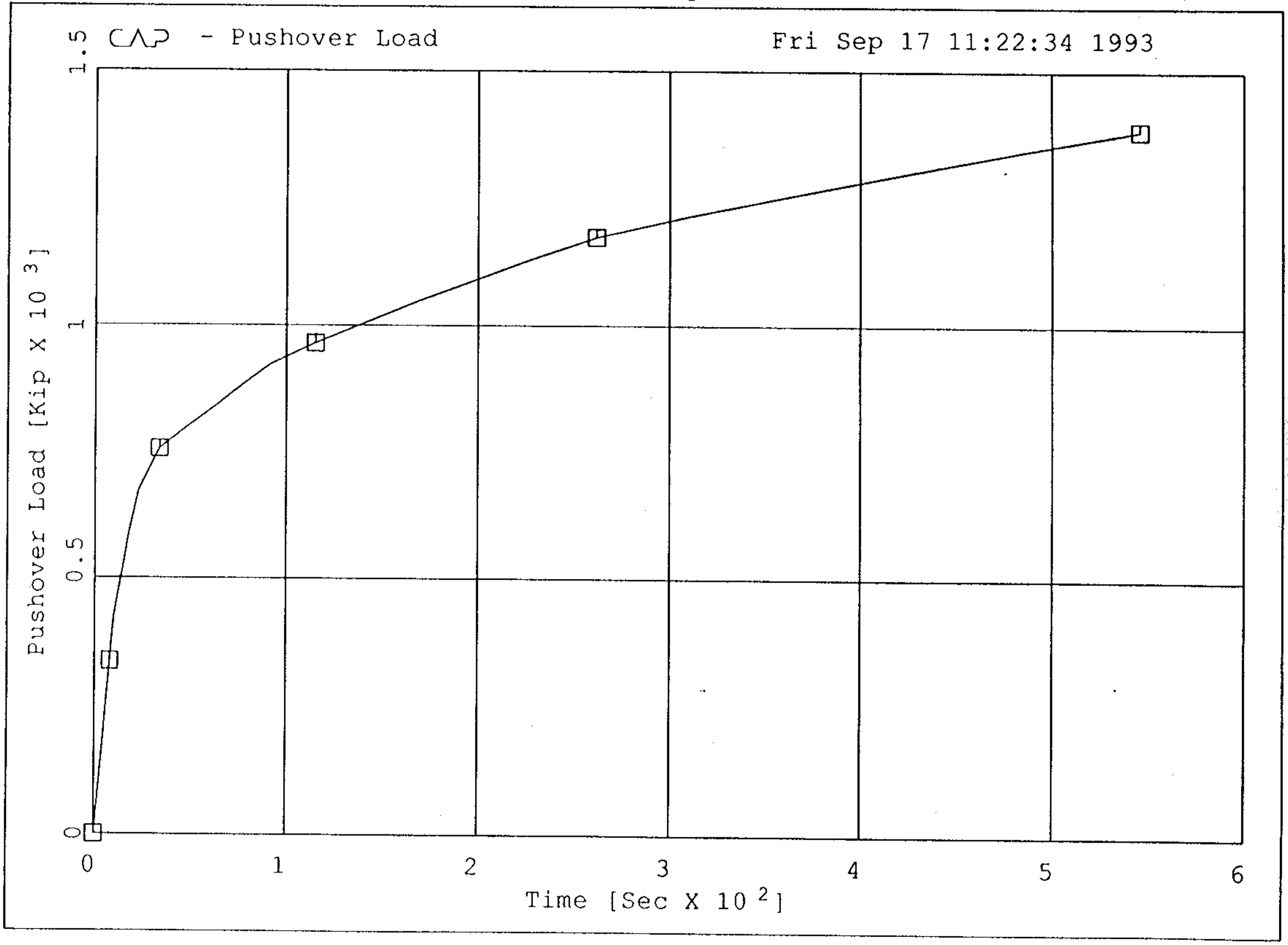
PLATFORM SS139 (T25)

This Platform has Extensive Damages Against Hurricane Andrew Loads

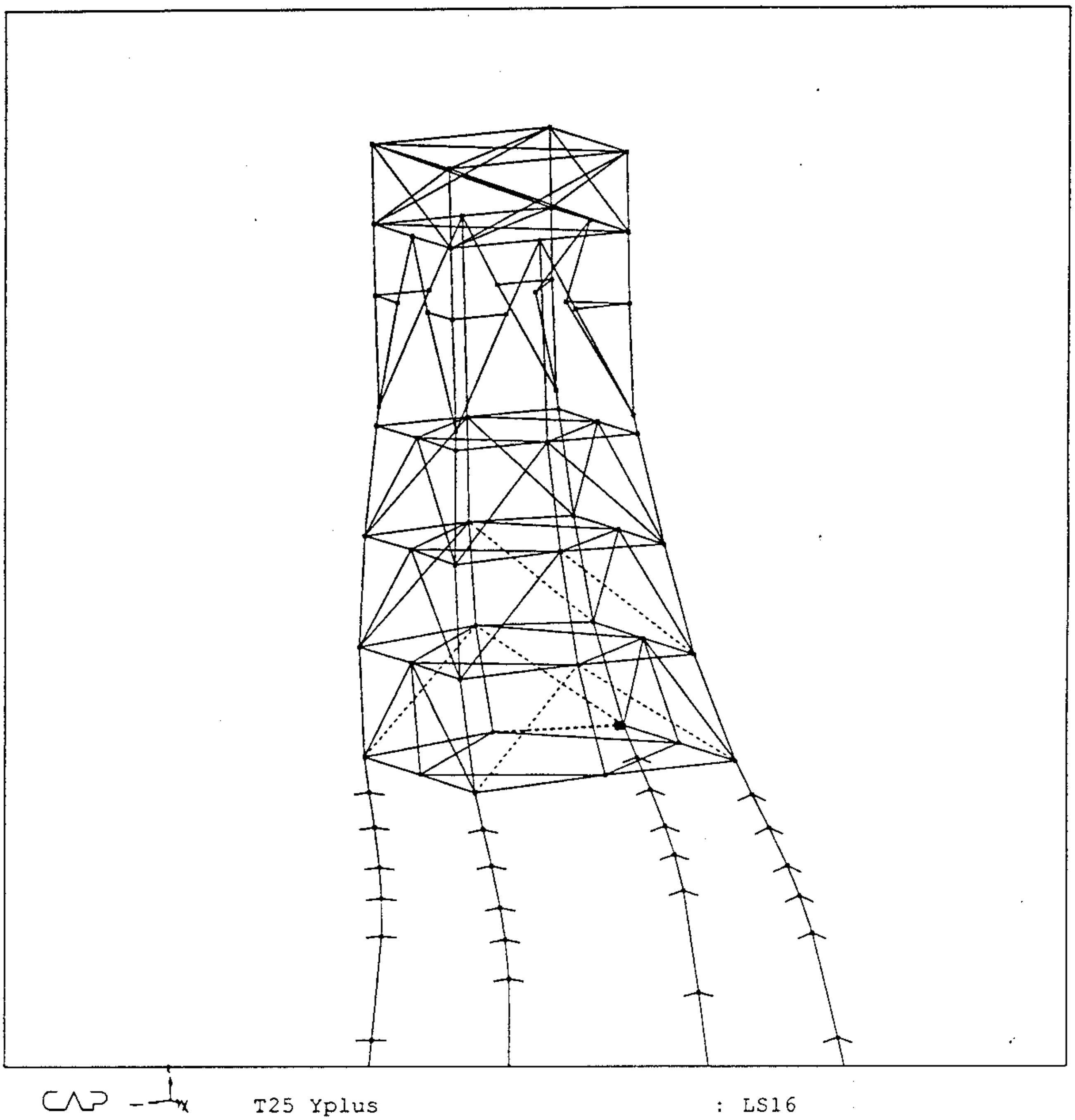
Water Depth = 62.00 ft.

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.711 * Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossin Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	U Platform (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Orthogonal-1 Direction			(235.5 deg. to 280.5 deg.)											1342.00
1	245.9	13.88	23.74	8.82	6.53	0.135982	1.64537	2.40169	1.39191	1.11353	25.575	327.04		
2	246.3	15.72	26.89	9.44	6.99	0.135982	1.64537	2.40169	1.61311	1.29049	29.009	442.62		
3	248.1	17.68	30.25	14.67	10.86	0.959459	2.3864	1.83505	1.92556	1.54045	33.929	617.61		
4	248.1	19.59	33.51	14.69	10.87	0.959459	2.3864	1.83505	2.26672	1.81337	37.836	754.34		
5	243.1	21.82	37.33	11.12	8.23	0.959459	2.3864	1.83505	2.71504	2.17203	42.509	934.04		
6	238.1	25.39	43.44	12.01	8.89	0.959459	2.3864	1.83505	3.29726	2.63781	49.738	1246.07		
7	257.7	29.59	50.63	13.26	9.81	0.959459	2.3864	1.83505	4.25312	3.40249	58.748	1691.27		
Diagonal Direction			(280.5 deg. to 325.5 deg.)											
8	292.8	30.12	51.53	15.75	11.66	0.655227	2.1058	1.93542	4.12	3.4987	58.894	1746.70		
9	315.6	27.82	47.60	14.42	10.67	0.655227	2.1058	1.93542	2.93	2.49456	52.850	1416.48		
Orthogonal-2 Direction			(325.5 deg. to 370.5 deg.)											1342.00
10	328.4	26.21	44.85	13.15	9.73	0.959459	2.3864	1.83505	2.31651	1.85321	49.269	1224.58		
11	334.3	25.39	43.44	11.9	8.81	0.959459	2.3864	1.83505	1.953	1.5624	47.172	1130.63		
12	341.5	23.56	40.30	11.58	8.57	0.959459	2.3864	1.83505	1.27225	1.0178	42.729	942.95		
13	343.2	22.41	38.34	11.11	8.22	0.959459	2.3864	1.83505	0.68561	0.54849	39.645	821.82		
14	342.1	20.96	35.87	10.74	7.95	0.959459	2.3864	1.83505	0.25779	0.20623	36.358	701.15		
15	342.3	19.39	33.17	10.43	7.72	0.135982	1.64537	2.40169	0.18707	0.14966	33.418	621.75		
16	342.1	17.98	30.76	10.01	7.41	0.135982	1.64537	2.40169	0.43826	0.35061	31.335	532.71		
17	339.5	16.90	28.91	9.72	7.19	0.135982	1.64537	2.40169	0.61491	0.49193	29.716	468.96		
18	339.2	15.85	27.11	9.3	6.88	0.135982	1.64537	2.40169	0.54132	0.43306	27.823	400.39		

Project: T25 Model: Y-plus Version: 2

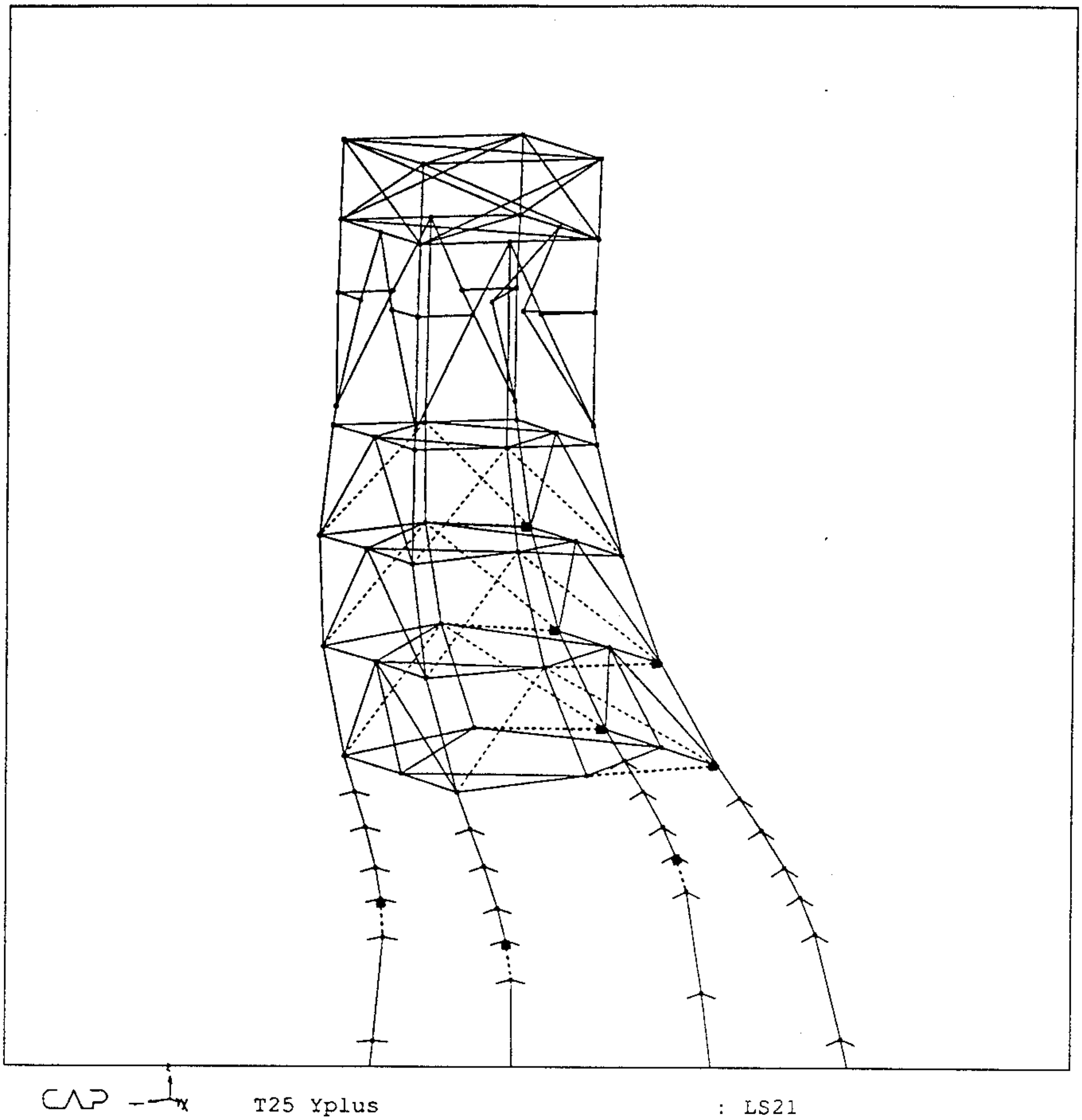


T25 Yplus



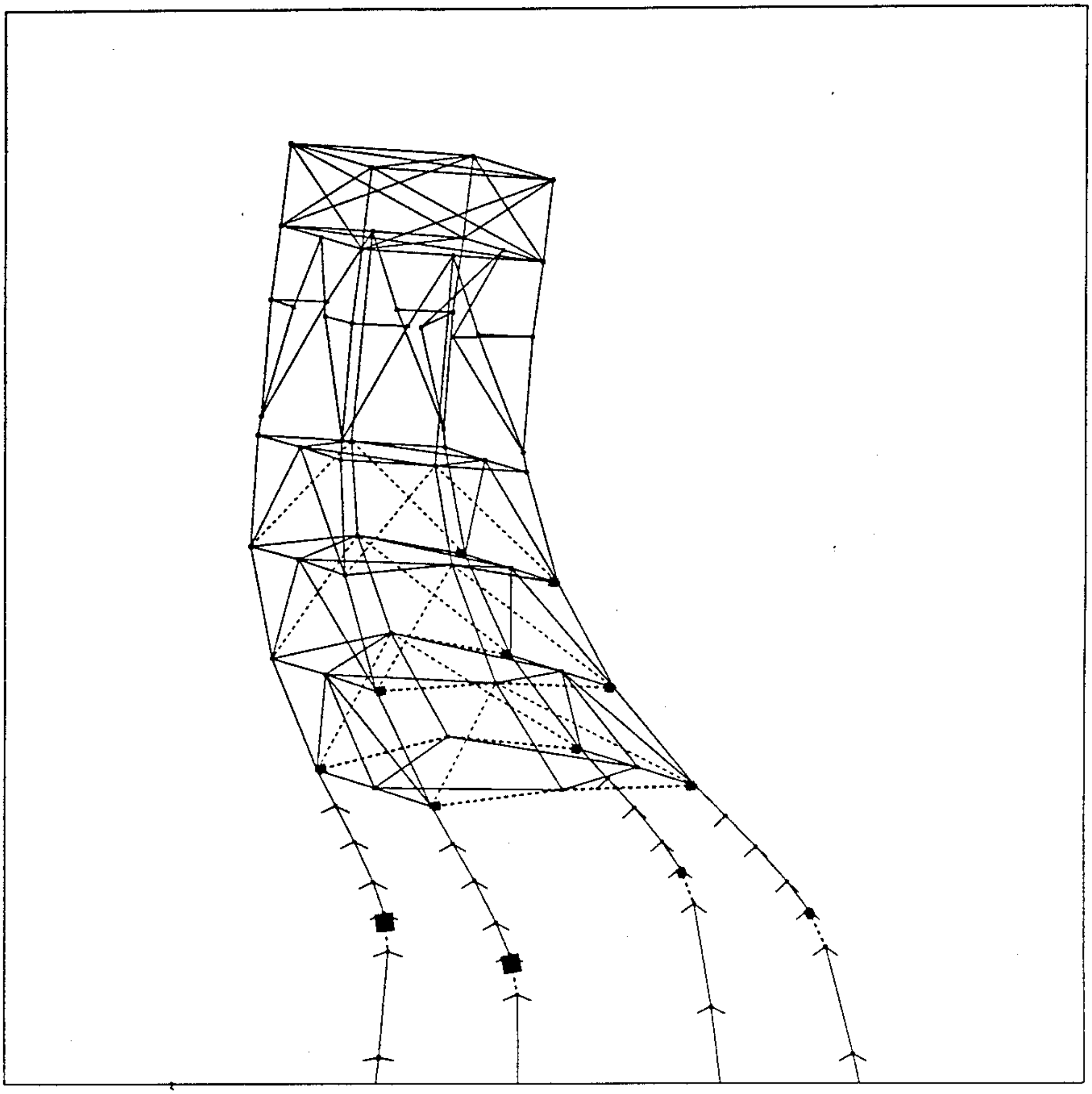
T25 Yplus : LS16

Project: T25 Model: Y-plus Version: 2



T25 Yplus : LS21

Project: T25 Model: Y-plus Version: 2



CAP

T25 Yplus LS24

Project: T25 Model: Y-plus Version: 2

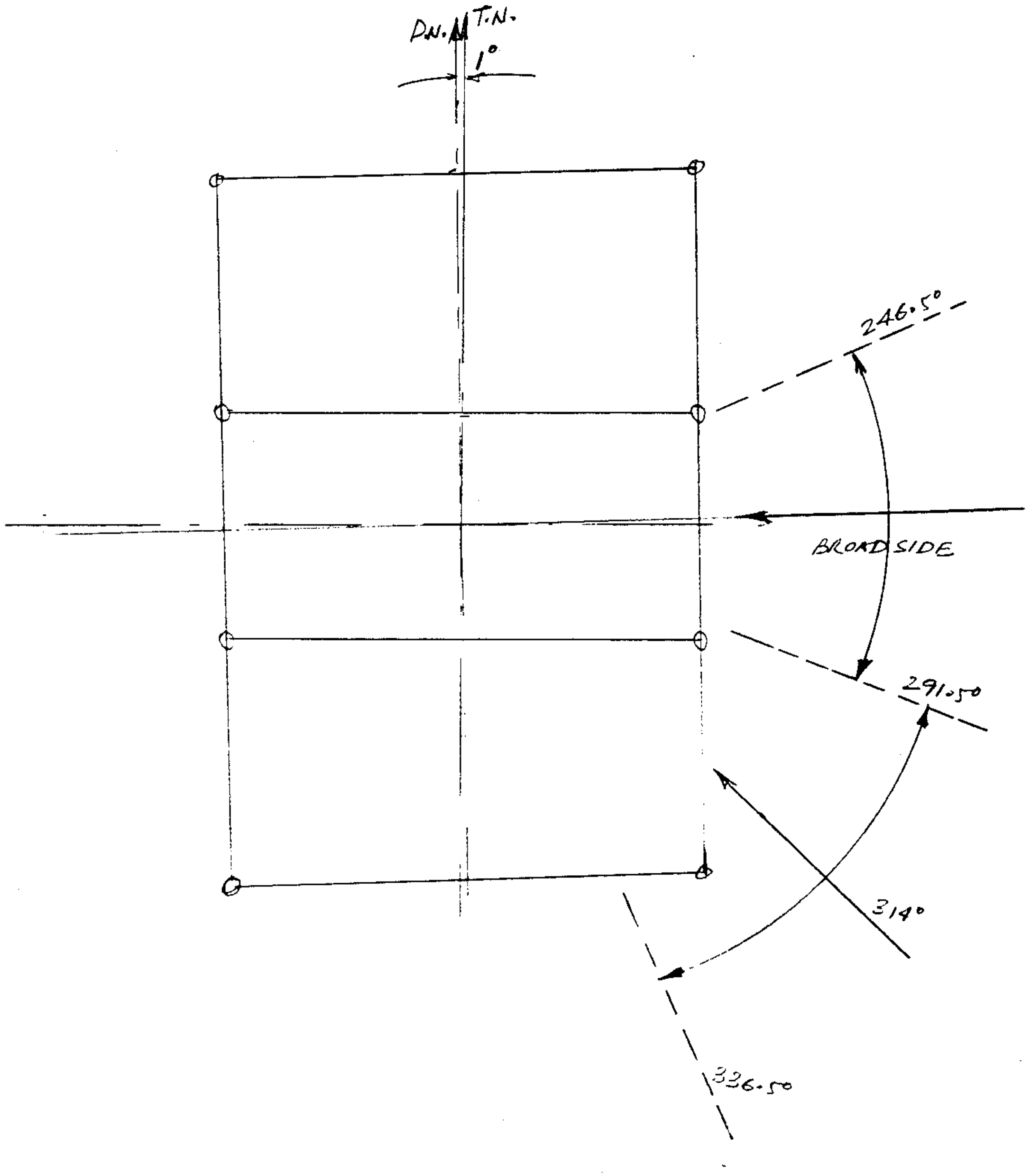
Platform ST161A

PMB ENGINEERING

By RKA Date / / Checked by Sheet No.

Project ANDREW - JIP Job No. 295

Subject PLATFORM STIGIA



Amoco Production Company

ENGINEERING CHART

FILE

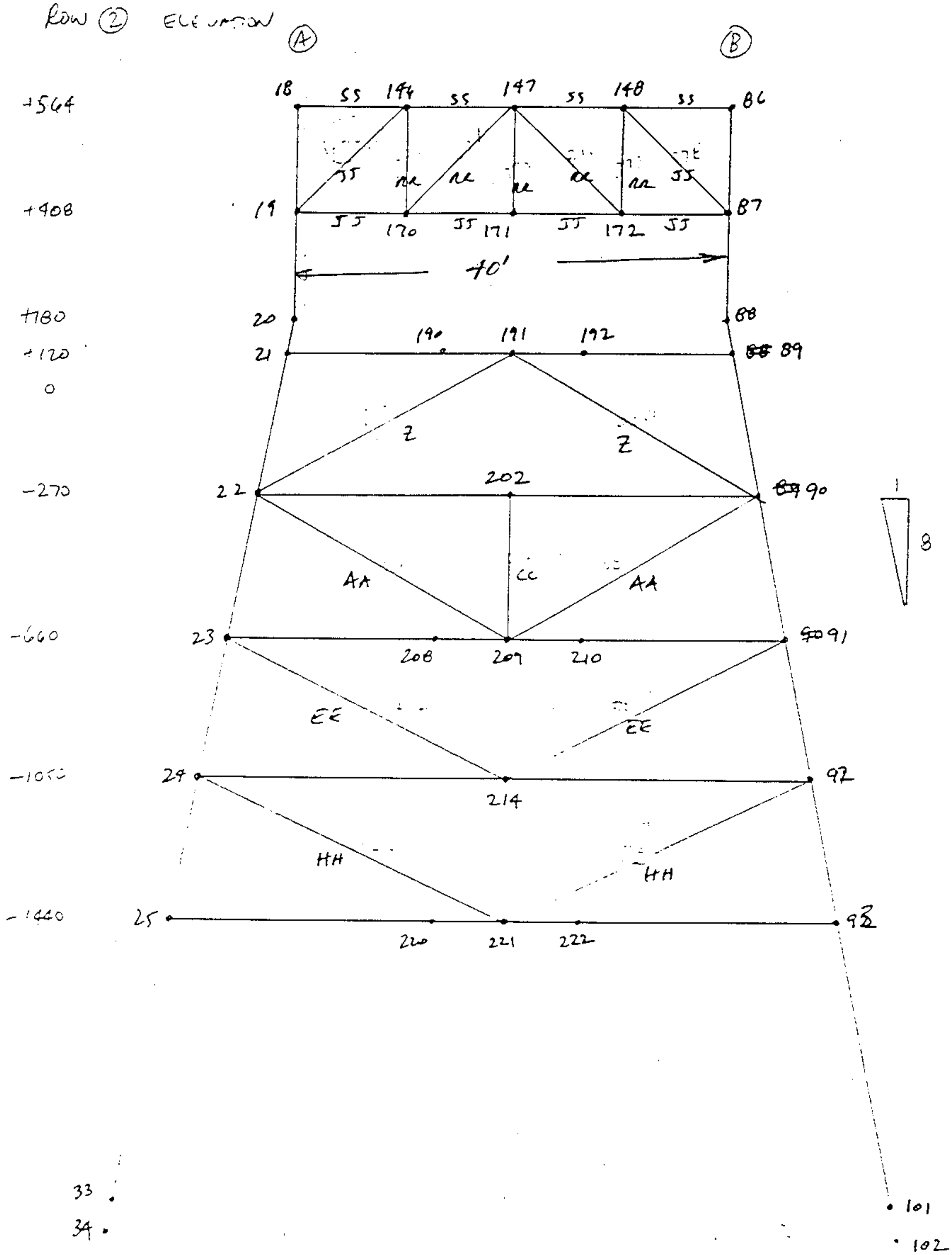
APPN

DATE

BY

SUBJECT STIGIA

Row (2) ELEVATION



Amoco Production Company

ENGINEERING CHART

FILE

APPN

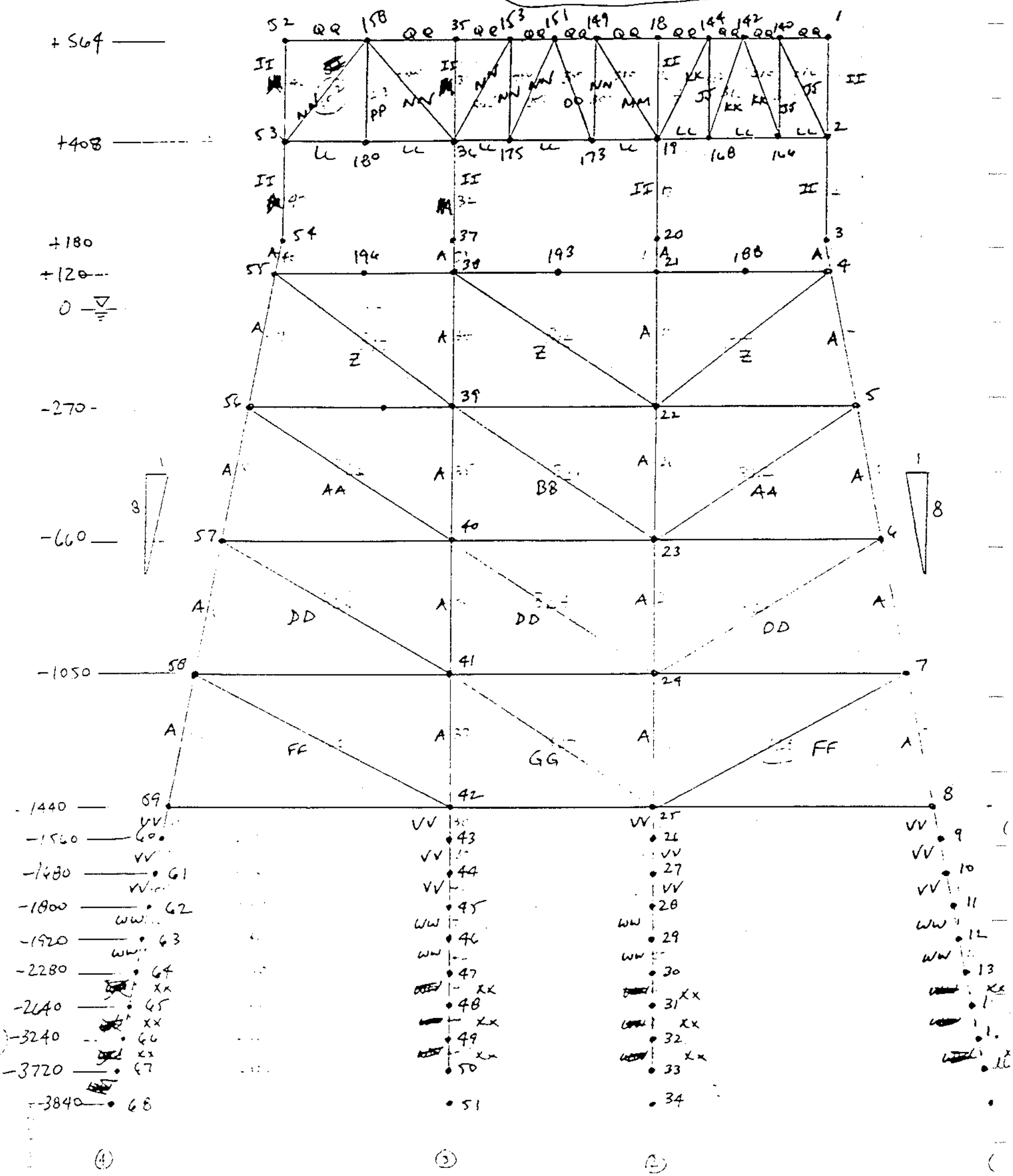
SUBJECT STIGIA

DATE

BY J.M. L

ELEVATION (A)

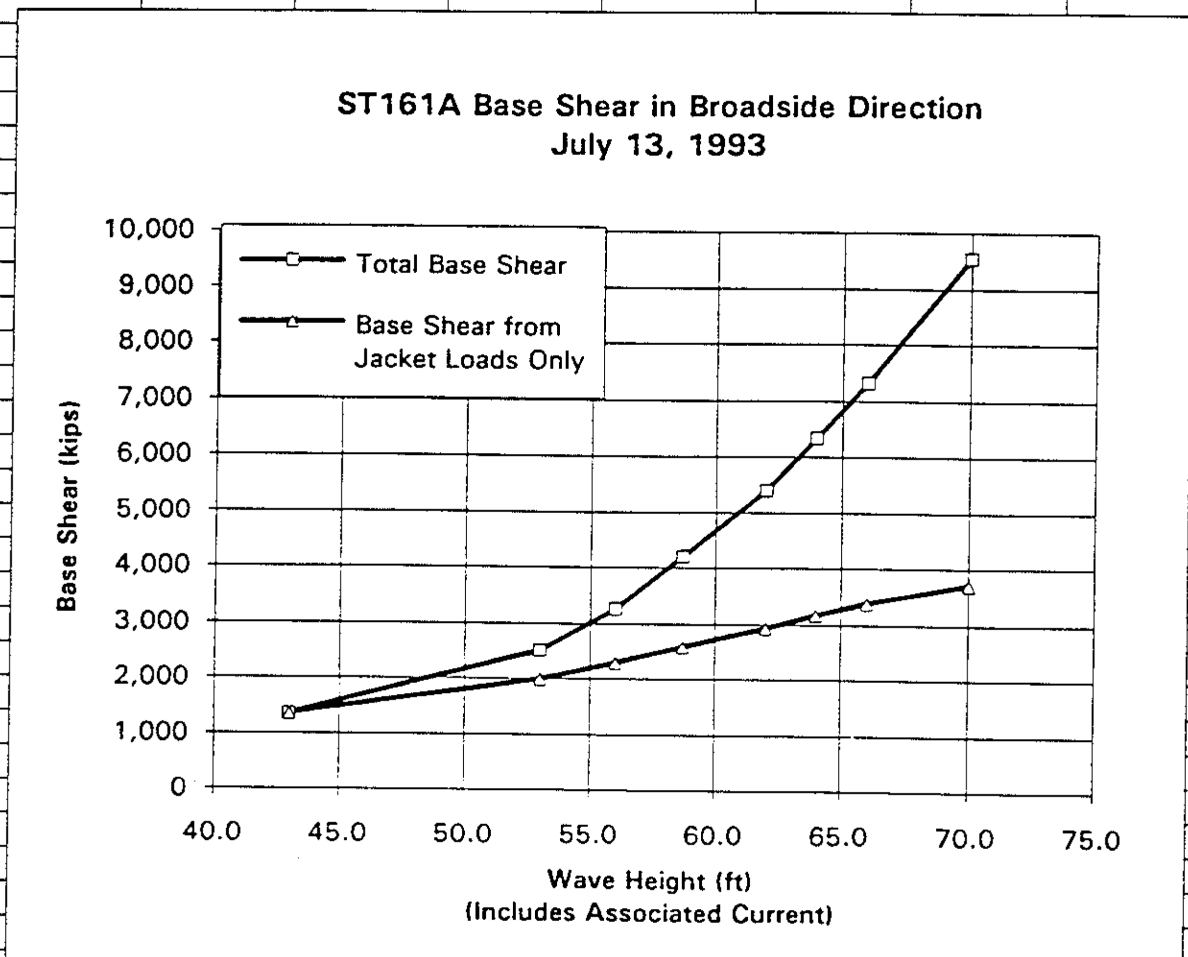
ATTACHMENT B



ST161A Base Shear in Broadside Direction
Numbers are from Gary Imm
13-Jul-93

Wave Height (ft)	Wave Period (sec)	Assoc Current (fps)	Wave in Deck Load (kips)	Total Base Shear (kips)		Calc. Jacket Load (kips)
70.0	16.0	4.5	5,832	9,547		3,715
66.0	15.5	4.4	3,941	7,319		3,378
64.0	15.0	4.3	3,170	6,332		3,162
62.0	14.5	4.1	2,475	5,400		2,925
58.7	13.9	3.8	1,620	4,200		2,580
56.0	13.0	3.6	980	3,263		2,283
53.0	12.0	3.5	512	2,505		1,993
43.0	11.0	3.2	0	1,346		1,346

Note: 58.7 ft. wave = max from Hurricane Andrew, based upon hindcast and observed damage

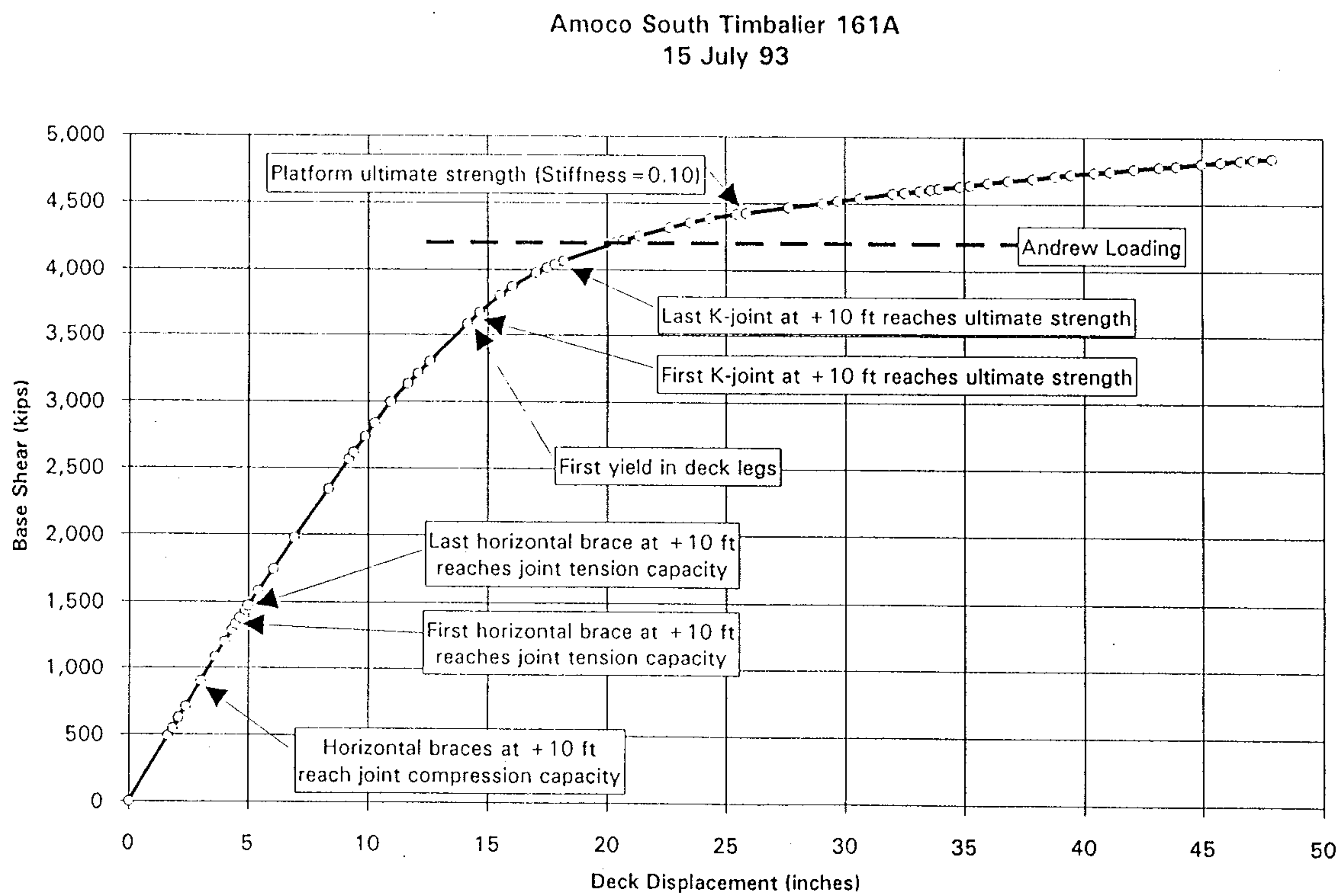


PLATFORM ST161A

Water Depth = 118 ft.

Platform Damaged During Hurricane Andrew

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.670 * Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossin Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Broadside Direction:			(246.5 deg. to 291.5 deg.)										
1	249.0	17.81	29.744	9.915	7.34	0.00508604	2.49805	3.18053	1.08	32.441	325.42		4426.00
2	249.5	20.63	34.456	10.726	7.94	0.00508604	2.49805	3.18053	1.36	37.857	531.77		
3	248.3	23.23	38.799	11.215	8.30	0.00508604	2.49805	3.18053	1.74	43.152	806.42		
4	248.9	26.18	43.725	11.891	8.80	5.09E-03	2.49805	3.18053	2.25	49.346	1235.45		
5	251.5	30.28	50.566	12.746	9.43	0.00508604	2.49805	3.18053	2.91	57.831	2046.39		
6	265.1	34.08	56.914	13.688	10.13	9.79E-06	1.28126	4.78314	3.56	61.472	3516.40		
7	290.4	35.02	58.495	14.270	10.56	9.7859E-06	1.28126	4.78314	3.56	63.061	3973.06		
Diagonal Direction:			(291.5 deg. to 336.5 deg.)										
8	311.6	31.82	53.144	13.447	9.95				2.87				Not Available
9	319.2	29.99	50.095	12.753	9.44				2.34				
10	320.4	28.33	47.314	11.996	8.88				1.90				
11	322.8	26.98	45.056	11.470	8.49				1.38				
12	325.5	25.44	42.497	11.230	8.31				1.08				
13	327.4	23.78	39.722	10.829	8.01				0.79				
14	328.9	22.35	37.336	10.558	7.81				0.48				
15	330.2	20.95	34.993	10.319	7.64				0.00				
16	331.3	19.61	32.752	10.027	7.42				0.00				



Hurricane Andrew
Loading = 4199 kips

Platform Ultimate
Strength = 4426 kips

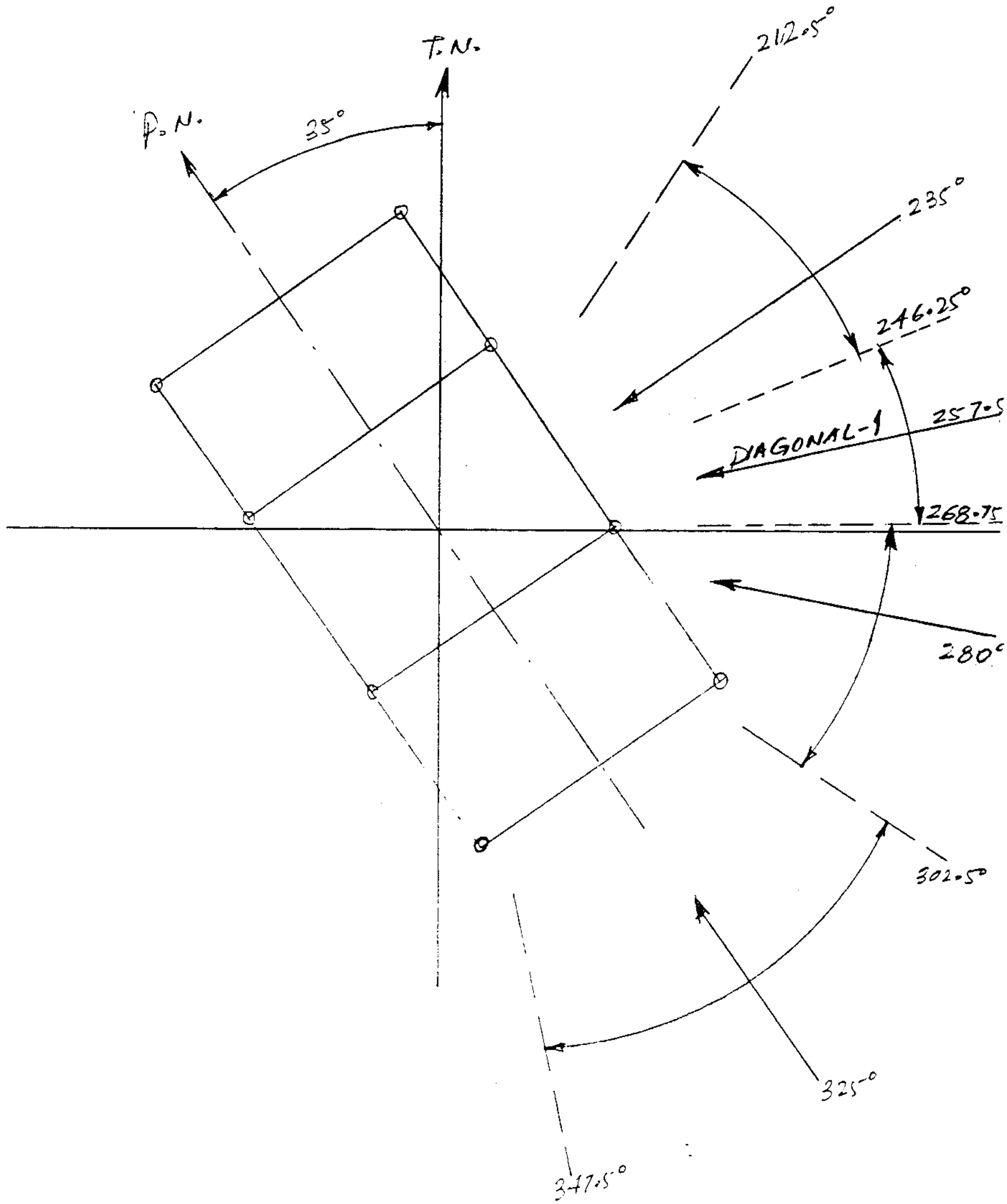
Platform ST177B

PMB ENGINEERING

By RKA Date 05/27/93 Checked by ____ Sheet No. ____

Project ANDREW JIP Job No. 295

Subject PLATFORM SS 177B



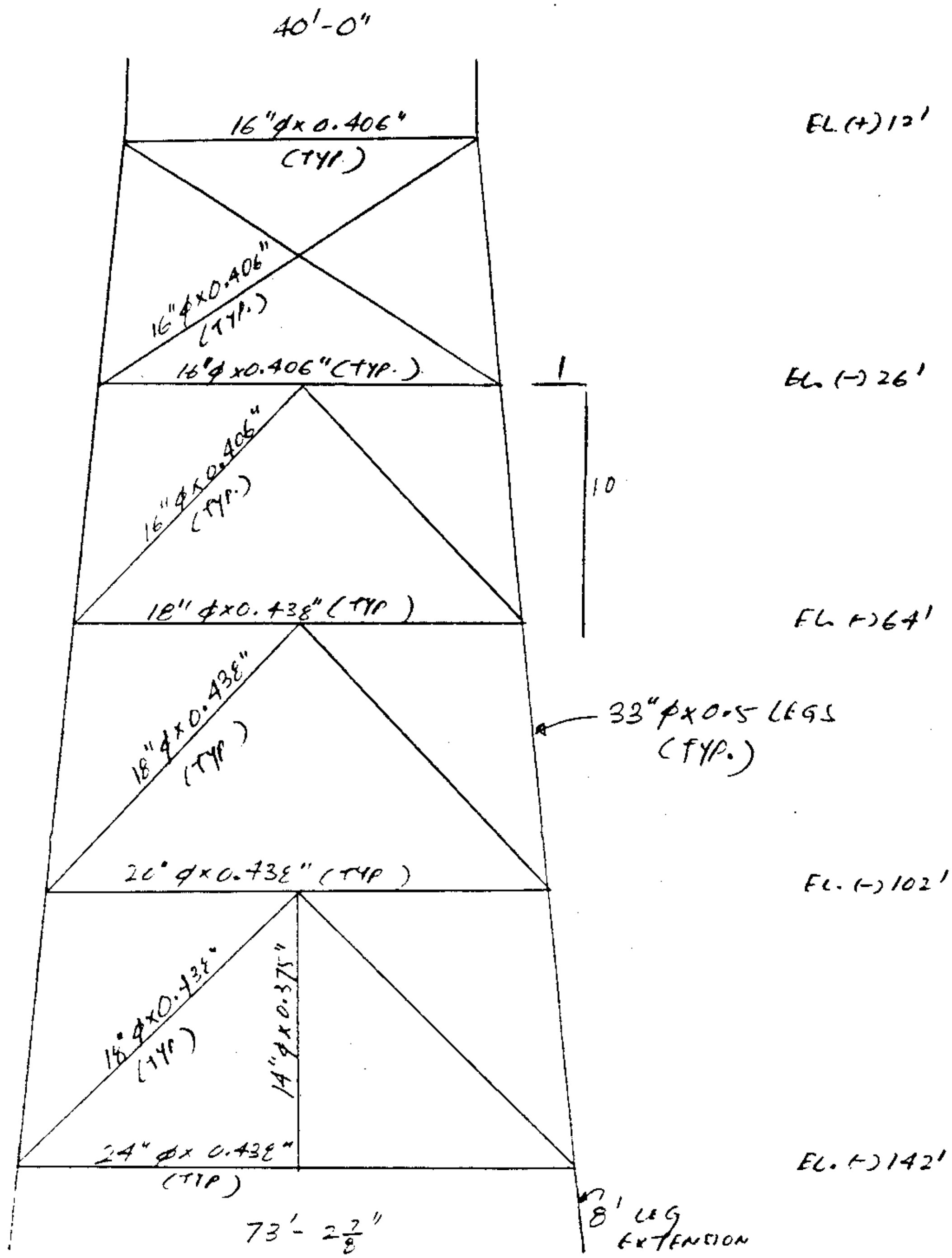
PMB ENGINEERING

By ________ Date / / Checked by ________ Sheet No. ________

Project ANDREW JIP Job No. 295

Subject PLATFORM STITTR.

MAIN DECK ELEV.(+)50'
CELLAR DECK ELEV.(+)39'



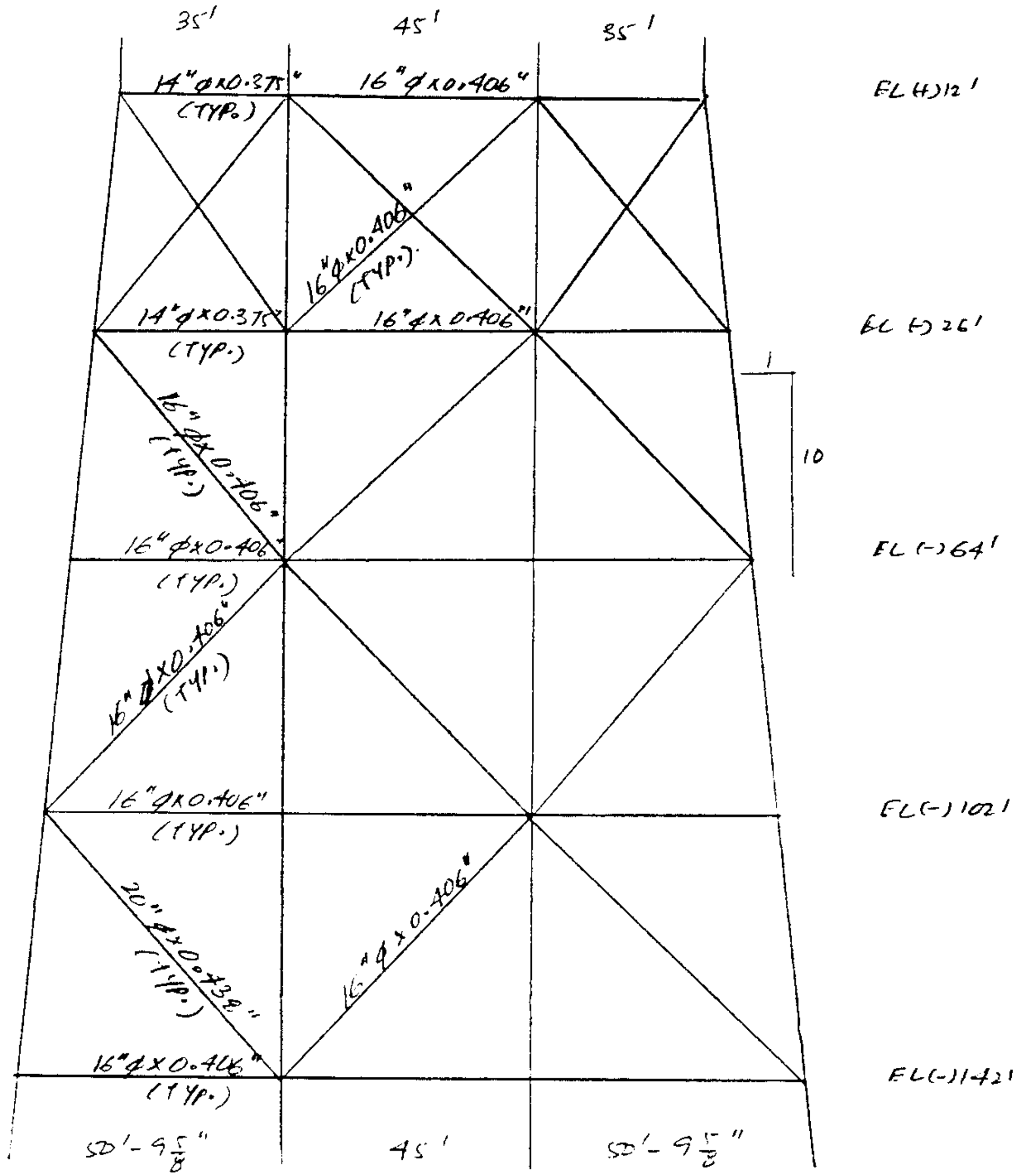
8-30"φ PILES WITH 187' PENETRA
12-30"φ CONDUCTORS

ROW 1-4

By______ Date / / ______ Checked by______ Sheet No.______

Project ANDREW JIP ______ Job No. 295

Subject PLATFORM ST177B



ROWS A & B

Chevron ST177B - C1, C2, C3 Values

Direction		End On			Broadside			Diagonal	
Wave Height (Feet)	20 to 50	50 to 60	60 to 70	20 to 50	50 to 60	60 to 70	20 to 50	50 to 60	60 to 70
C1	0.241864	0.064501	0.025502	0.613863	0.002239	0.027274	0.461187	0.016880	0.096126
C2	3.300310	2.825410	2.386020	4.050330	1.969570	2.526410	3.109980	1.990230	2.284390
C3	2.277020	2.612370	2.842470	2.064910	3.498940	2.917420	2.136450	2.983740	2.566930

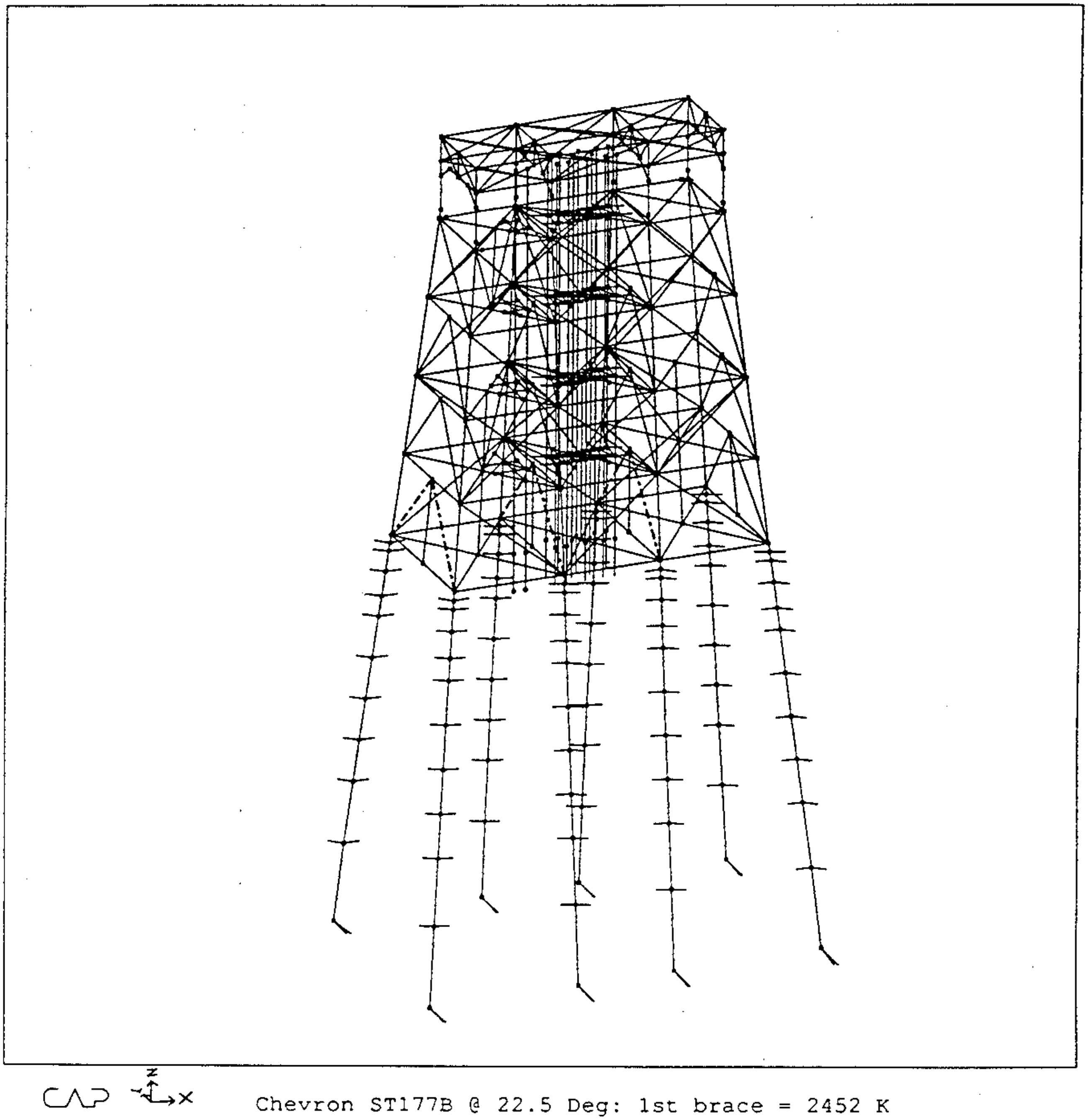
PLATFORM ST177B

Water Depth = 142 ft.

This Platform Damaged Severely During Hurricane Andrew Loads

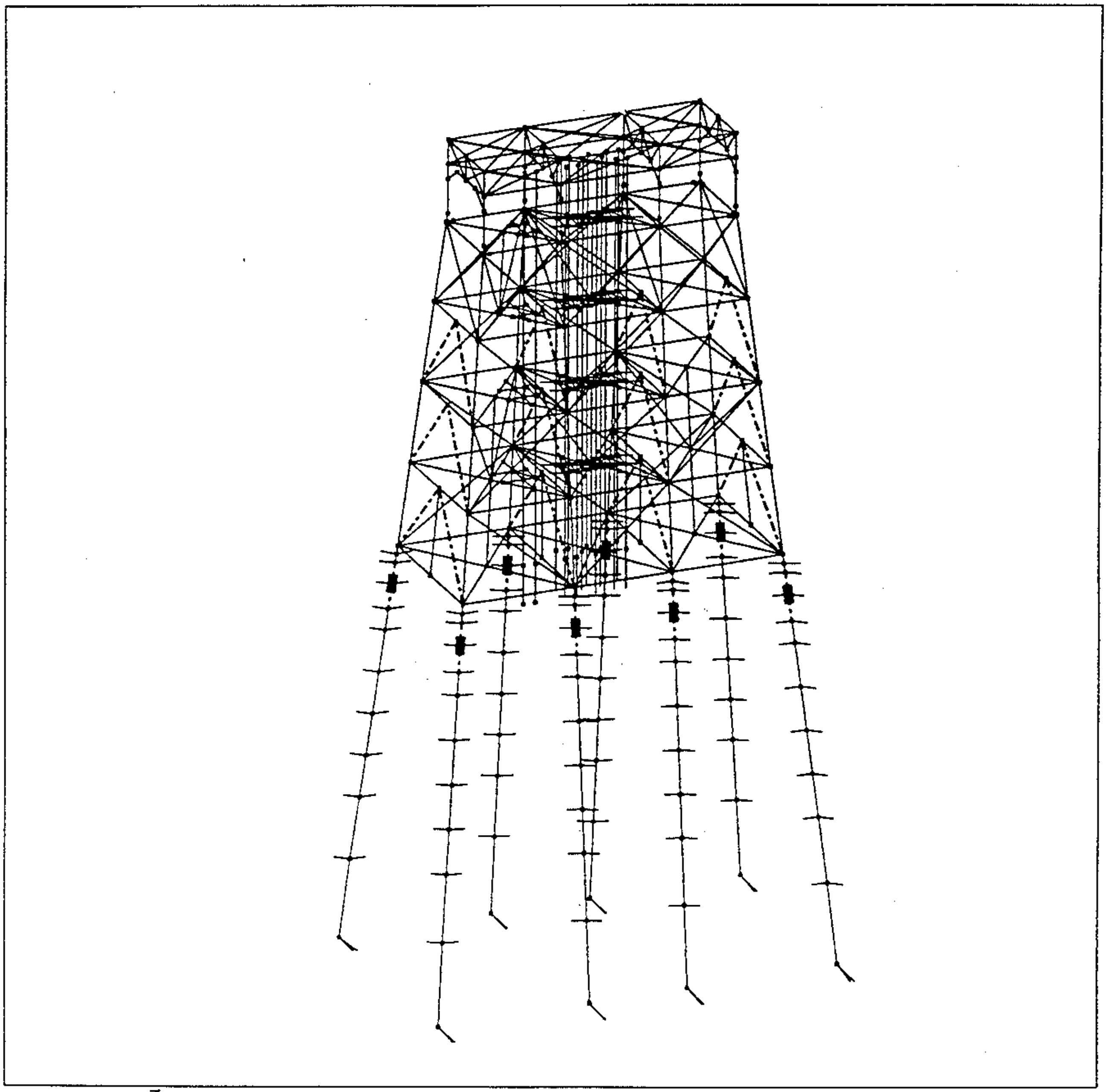
To be salvaged

Storm Hour	Wave Direction (Degree)	H max = Hs * 1.671 (ft.)	Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	U Platform (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Diagonal_1 Direction			(246.25 deg. to 268.75 deg.)				C1 - C3 for broadside used							4168.00
1	248.9	31.543	18.88	10.236	7.57	0.613863	4.05033	2.06491	1.00	0.82	34.881	940.54		
2	248.5	36.663	21.94	10.933	8.09	0.613863	4.05033	2.06491	1.28	1.06	40.937	1309.03		
3	246.9	41.718	24.97	11.622	8.60	0.613863	4.05033	2.06491	1.67	1.37	47.286	1762.94		
4	246.0	47.336	28.33	12.124	8.97	0.613863	4.05033	2.06491	2.19	1.81	54.669	2378.80		
5	249.1	55.215	33.04	13.090	9.69	0.002239	1.96957	3.49894	2.89	2.38	59.905	3709.27		
6	267.6	60.167	36.01	13.780	10.20	0.002239	1.96957	3.49894	3.46	2.86	65.797	5150.53		
Diagonal_2 Direction			(268.75 deg. to 302.5 deg.)											4378.00
7	298.6	57.915	34.66	13.740	10.17	0.01688	1.99023	2.98374	2.98	2.53	62.957	3937.83		
End-On Direction			(302.5 deg. to 347.5deg.)											4220.00
8	318.1	51.770	30.98	12.714	9.41	0.06450	2.82541	2.61237	2.21	1.54	56.135	2394.49		
9	323.2	49.551	29.65	12.216	9.04	0.24186	3.30031	2.27702	1.96	1.37	54.079	2136.55		
10	321.9	47.479	28.41	11.537	8.54	0.24186	3.30031	2.27702	1.82	1.28	51.695	1928.08		
11	321.7	45.382	27.16	11.162	8.26	0.24186	3.30031	2.27702	1.55	1.08	48.955	1703.25		
12	323.1	42.782	25.60	10.922	8.08	0.24186	3.30031	2.27702	1.33	0.93	45.844	1466.68		
13	325.3	40.025	23.95	10.647	7.88	0.24186	3.30031	2.27702	1.08	0.75	42.511	1235.09		
14	326.7	37.780	22.61	10.428	7.72	0.24186	3.30031	2.27702	0.82	0.58	39.677	1055.55		
15	327.5	35.334	21.15	10.137	7.50	0.24186	3.30031	2.27702	0.00	0.00	35.334	810.68		
16	328.5	32.808	19.63	9.886	7.32	0.24186	3.30031	2.27702	0.00	0.00	32.808	684.69		
17	328.4	32.178	19.26	9.785	7.24	0.24186	3.30031	2.27702	0.00	0.00	32.178	655.11		



Chevron ST177B @ 22.5 Deg: 1st brace = 2452 K

Project: ChevST177B Model: push225 Version: 1



Chevron ST177B @ 22.5 Deg: Fcap = 4168 Kips
Project: ChevST177B Model: push225 Version: 1

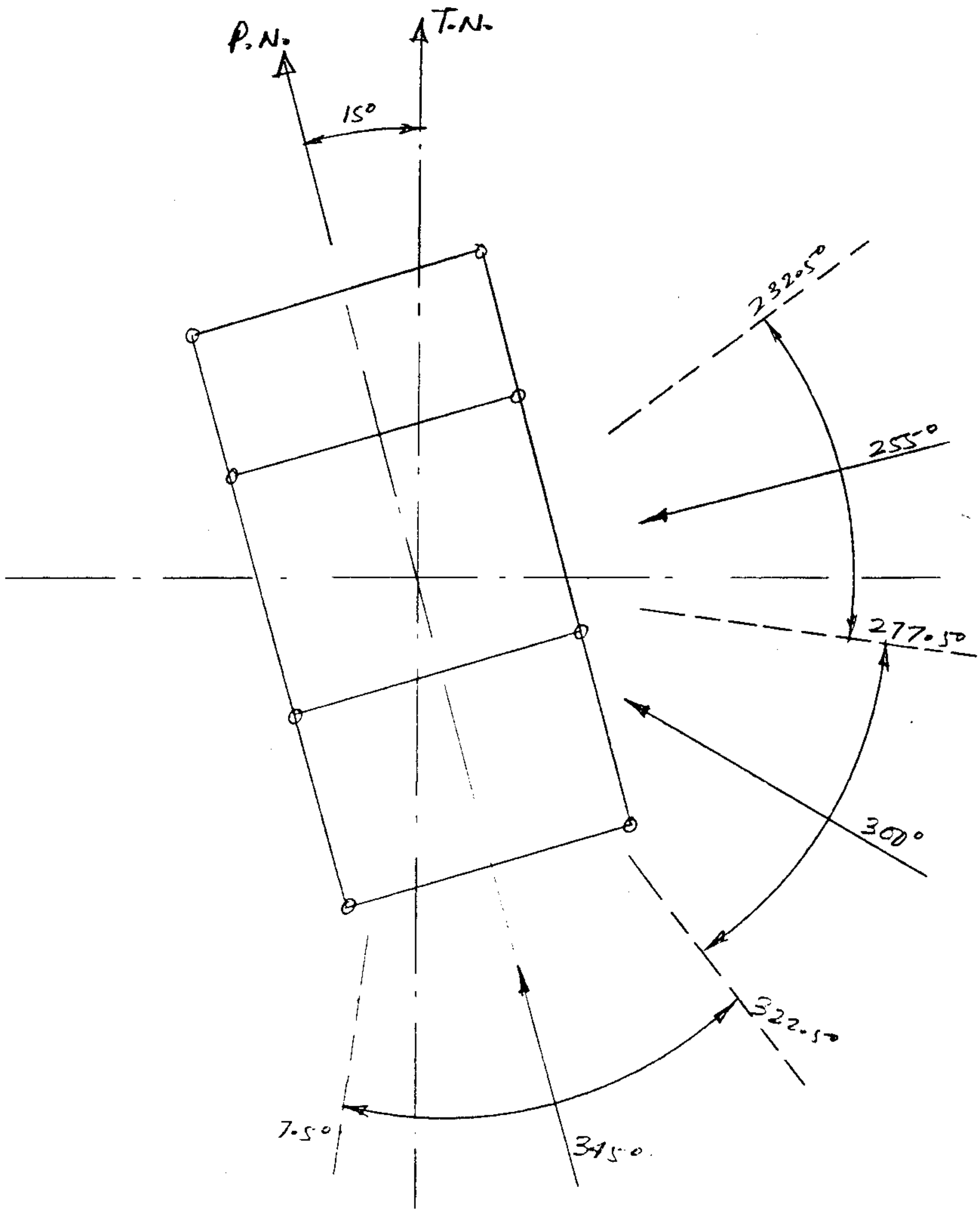
Platform ST151H

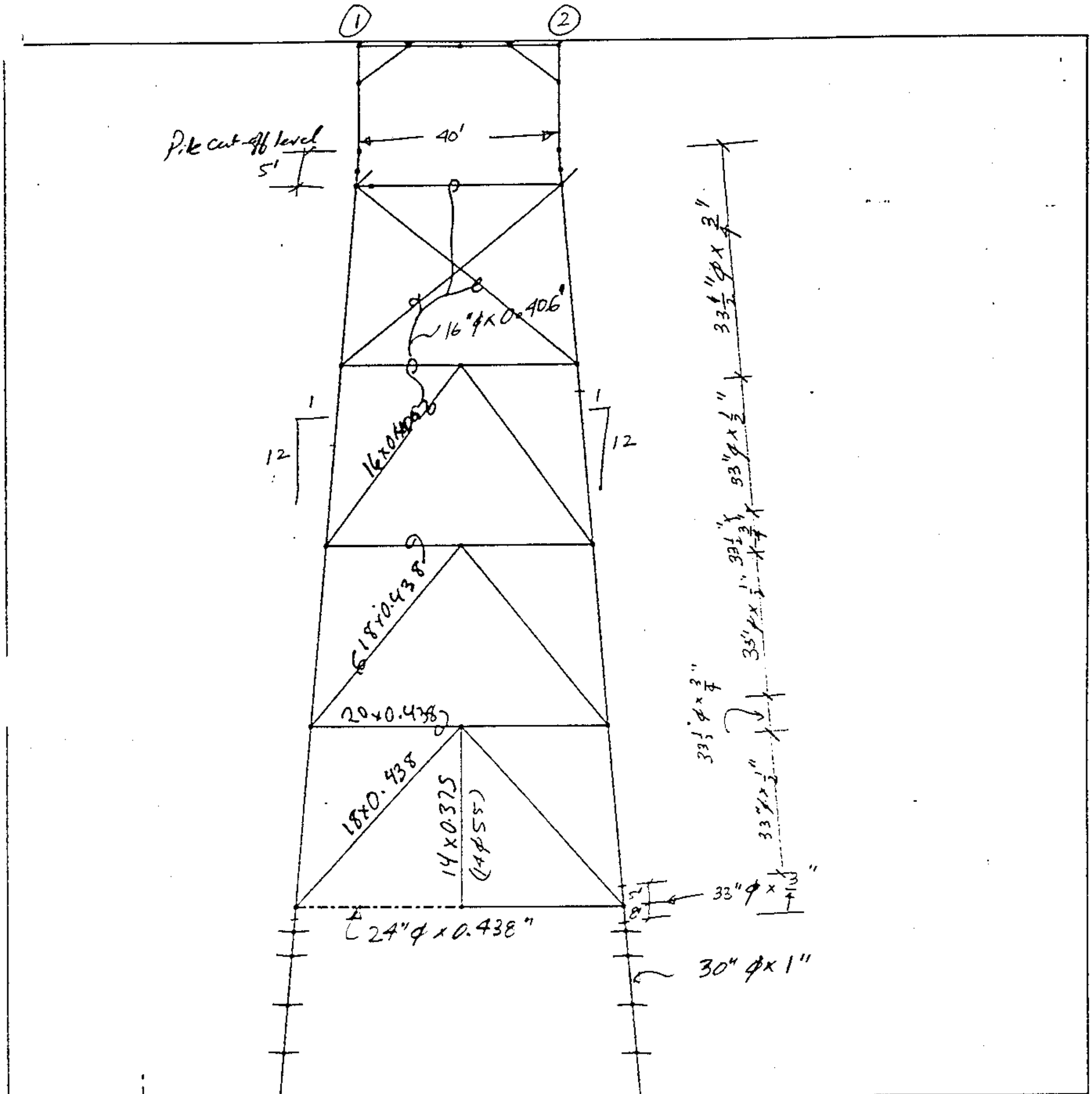
PMB ENGINEERING

By RKA Date / / Checked by ____ Sheet No. ____

Project ANDREW JIP Job No. 295

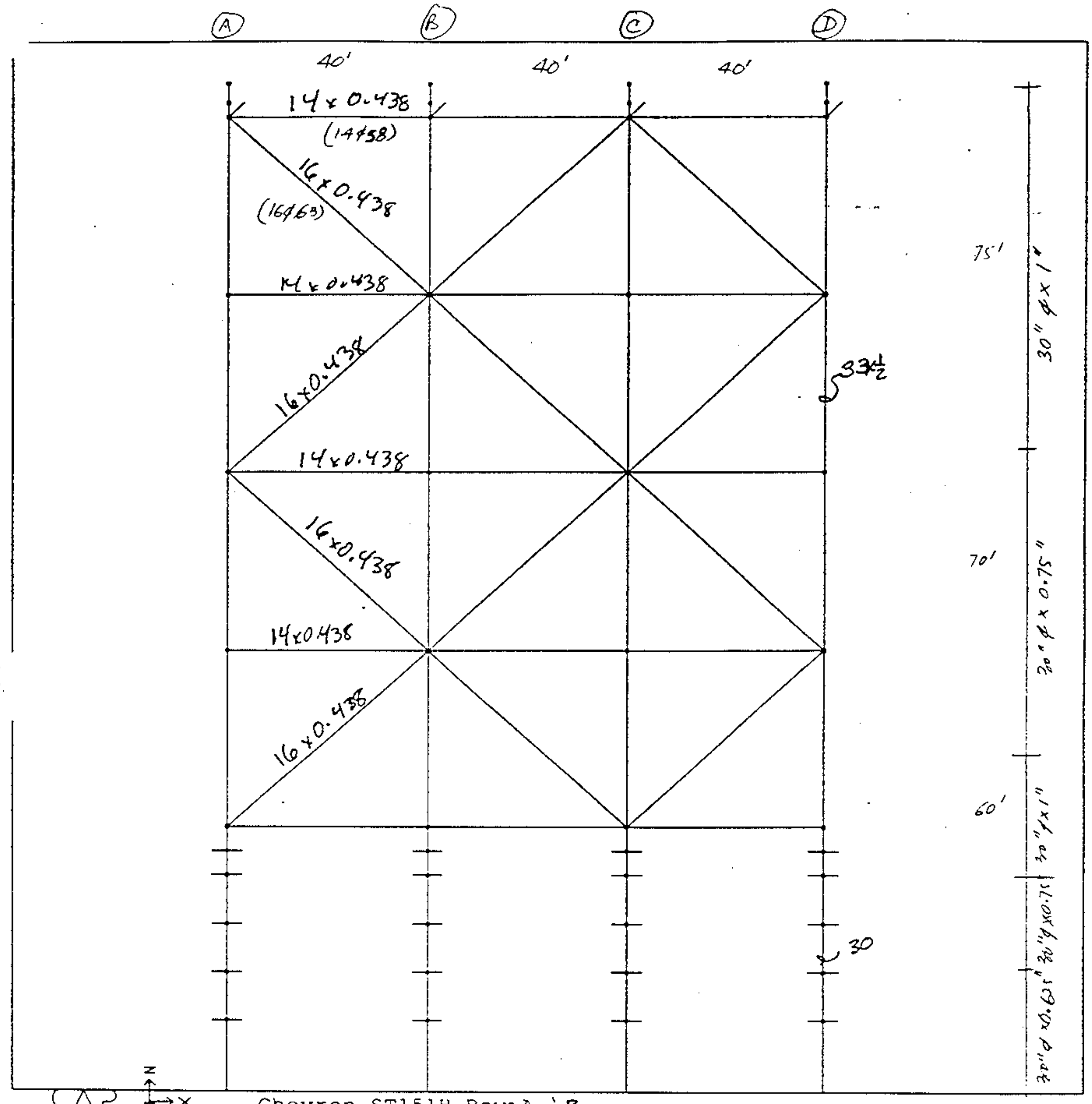
Subject PLATFORM ST 151A





Chevron ST151H Rows 1-4

Project: ChevST151H Model: pushx Version: 1



Chevron ST151H Rows A : B

Project: ChevST151H Model: pushx Version: 1

PILE

180' Penetration

WAVE DIRECTION DEG	FIRST WAVE HGT FT	LAST WAVE HGT FT	******* COEFFICIENTS ******* C1	C2	C3	
351	35	59	1.92E-01	5.42E+00	2.29E+00	End-on
351	60	70	3.65E-02	4.35E+00	2.72E+00	
351	71	75	3.65E-04	2.97E+00	3.82E+00	
36	35	59	2.16E-01	6.31E+00	2.28E+00	Diagonal
36	60	70	2.95E-02	5.05E+00	2.78E+00	
36	71	75	1.58E-05	2.93E+00	4.56E+00	
81	35	59	2.78E-01	5.94E+00	2.24E+00	Broadside
81	60	70	5.68E-03	3.84E+00	3.21E+00	
81	71	75	1.48E-19	1.02E+00	1.22E+01	

New wave direction will be 180°, 225 & 270°

FIGURE 4.2.3-1: LOAD PARAMETERS FOR PLATFORM ST151H

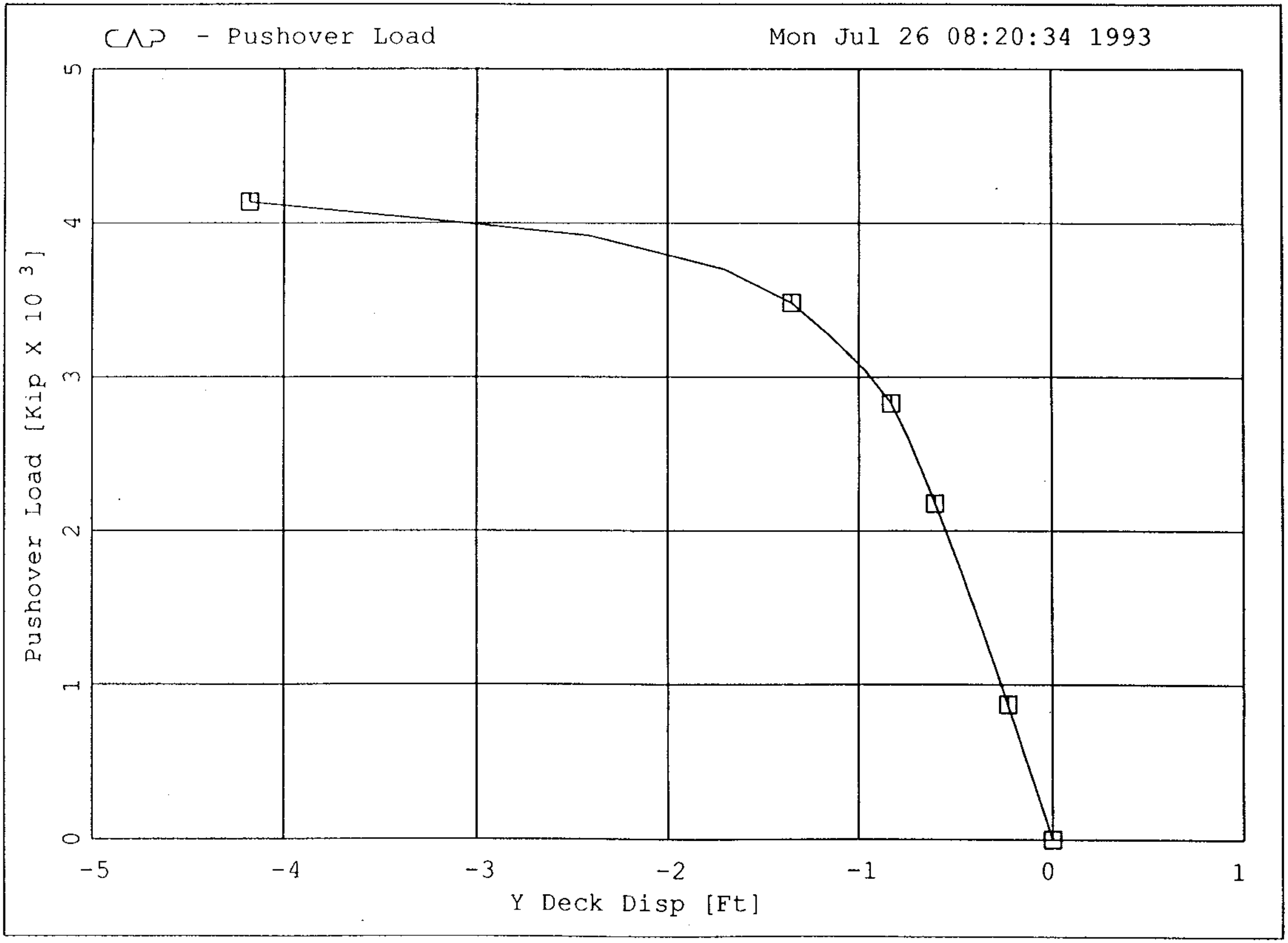
PLATFORM ST151H

This Platform Collapsed Against Hurricane Andrew Loads

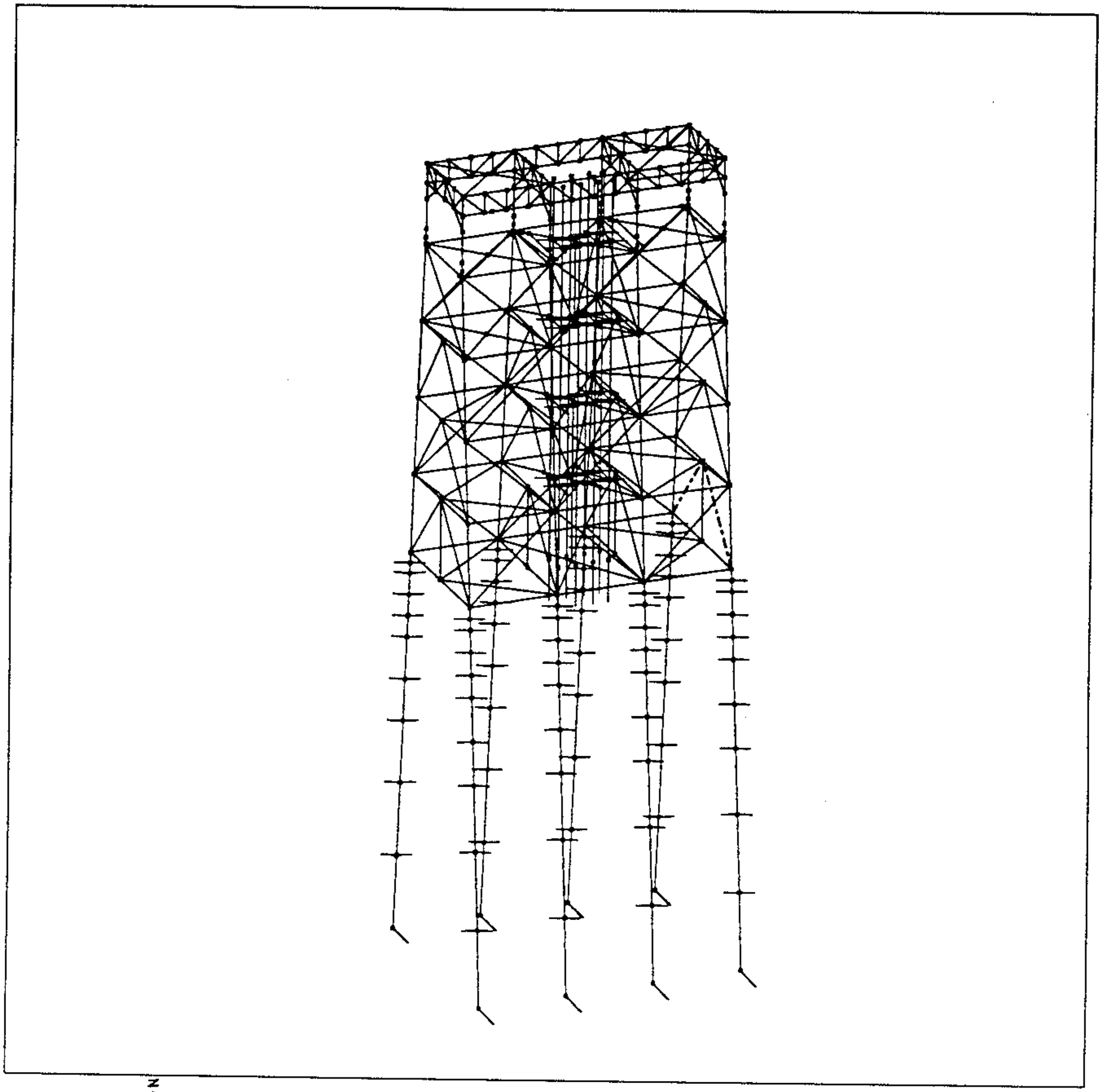
Water Depth = 137 ft.

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max = Hs * 1.683 (ft.)	Peak Spectral T Tp (sec.)	Zero Crossing Period Tz= 0.74* Tp (sec.)	U (knots)	C1	C2	C3	H+C2*U	BS (Kips)	Remarks	R median (Kips)
Broad-Side Direction			**(232.5 deg. to 277.5 deg.)**										4156.00
1	251.9	18.11	30.476	9.94	7.35	0.60	0.278	5.937	2.238	34.017	744.72		
2	253.3	20.79	34.984	10.88	8.05	0.75	0.278	5.937	2.238	39.427	1036.18		
3	252.9	23.18	39.011	11.25	8.32	0.95	0.278	5.937	2.238	44.658	1369.37		
4	255.0	25.96	43.682	11.91	8.81	1.21	0.278	5.937	2.238	50.869	1832.68		
5	258.3	29.64	49.882	12.72	9.42	1.53	0.278	5.937	2.238	58.974	2551.45		
6	268.2	33.89	57.029	13.67	10.12	1.87	0.278	5.937	2.238	68.110	3521.86		
Diagonal-1 Direction			**(277.5 deg. to 322.5 deg.)**										3999.00
7	286.4	36.16	60.854	14.45	10.69	2.04	0.02945	5.045	2.783	71.161	4206.09	Wave in Deck	
8	305.8	33.45	56.288	13.85	10.25	1.77	0.2155	6.306	2.275	67.428	3119.35		
9	314.2	31.06	52.277	12.95	9.58	1.34	0.2155	6.306	2.275	60.752	2460.64		
10	317.7	28.97	48.762	12.25	9.06	0.92	0.2155	6.306	2.275	54.560	1926.80		
11	321.6	27.50	46.278	11.76	8.70	0.52	0.2155	6.306	2.275	49.572	1549.23		
End_On Direction			**(322.5 deg. to 367.5 deg.)**										3450.00
12	324.9	25.79	43.410	11.40	8.44	0.31	0.1923	5.422	2.286	45.068	1160.73		
13	327.4	24.01	40.412	10.92	8.08	0.12	0.1923	5.422	2.286	41.039	937.02		
14	329.2	22.54	37.927	10.67	7.90	0.02	0.1923	5.422	2.286	38.011	786.40		
15	330.8	21.20	35.681	10.52	7.78	0.00	0.1923	5.422	2.286	35.681	680.53		
16	331.7	19.98	33.620	10.25	7.58	0.00	0.1923	5.422	2.286	33.620	594.00		
17	331.7	18.66	31.409	9.87	7.30	0.00	0.1923	5.422	2.286	31.409	508.46		

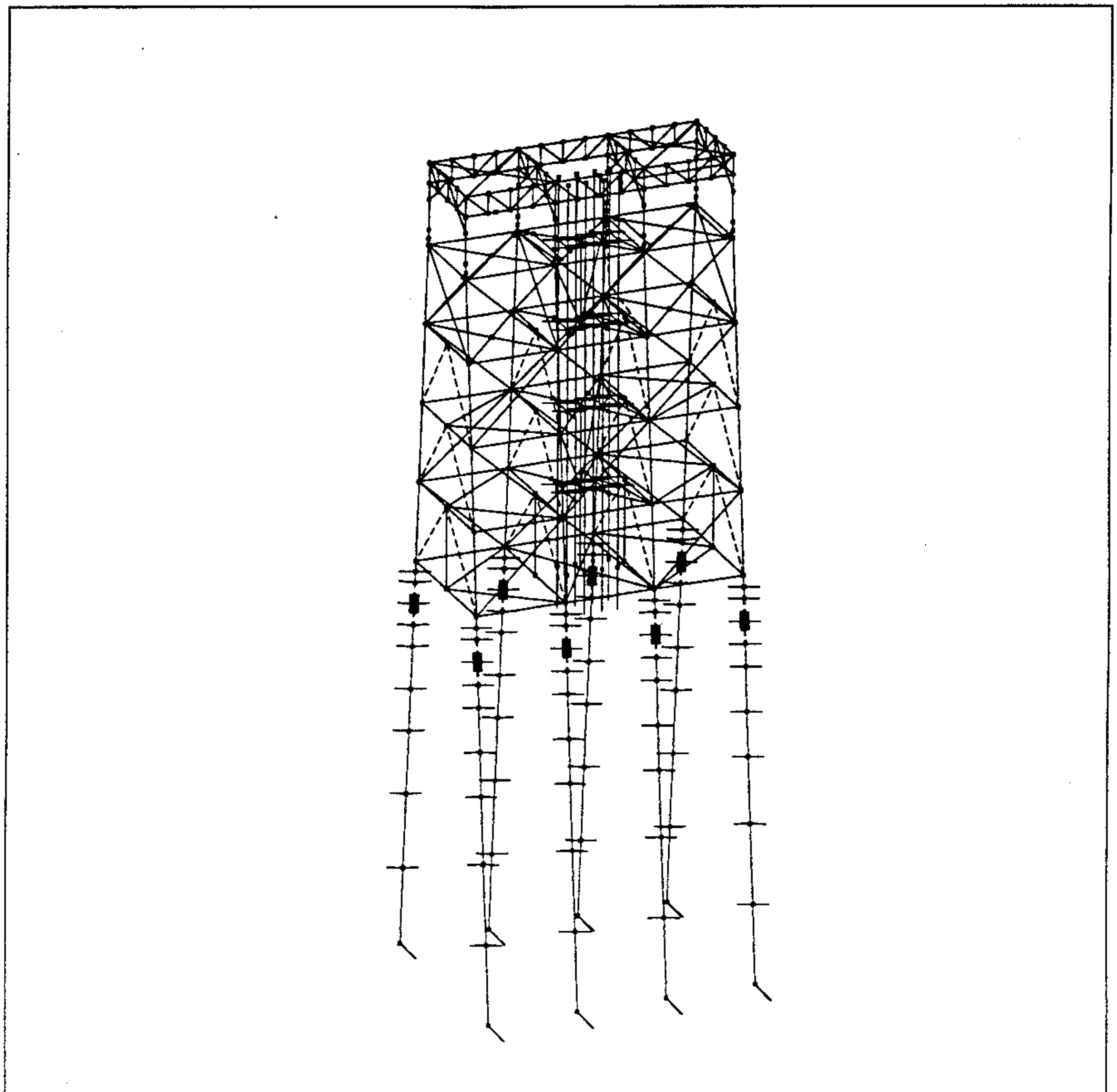
Project: ChevST151H Model: push225 Version: 1



Chevron ST151H @ 22.5 Deg



Chevron ST151H @ 22.5 Deg:First Brace = 2666 Kips
Project: ChevST151H Model: push225 Version: 1



Chevron ST151H @ 22.5 Deg: Fcap = 3999 kips

Project: ChevST151H Model: push225 Version: 1

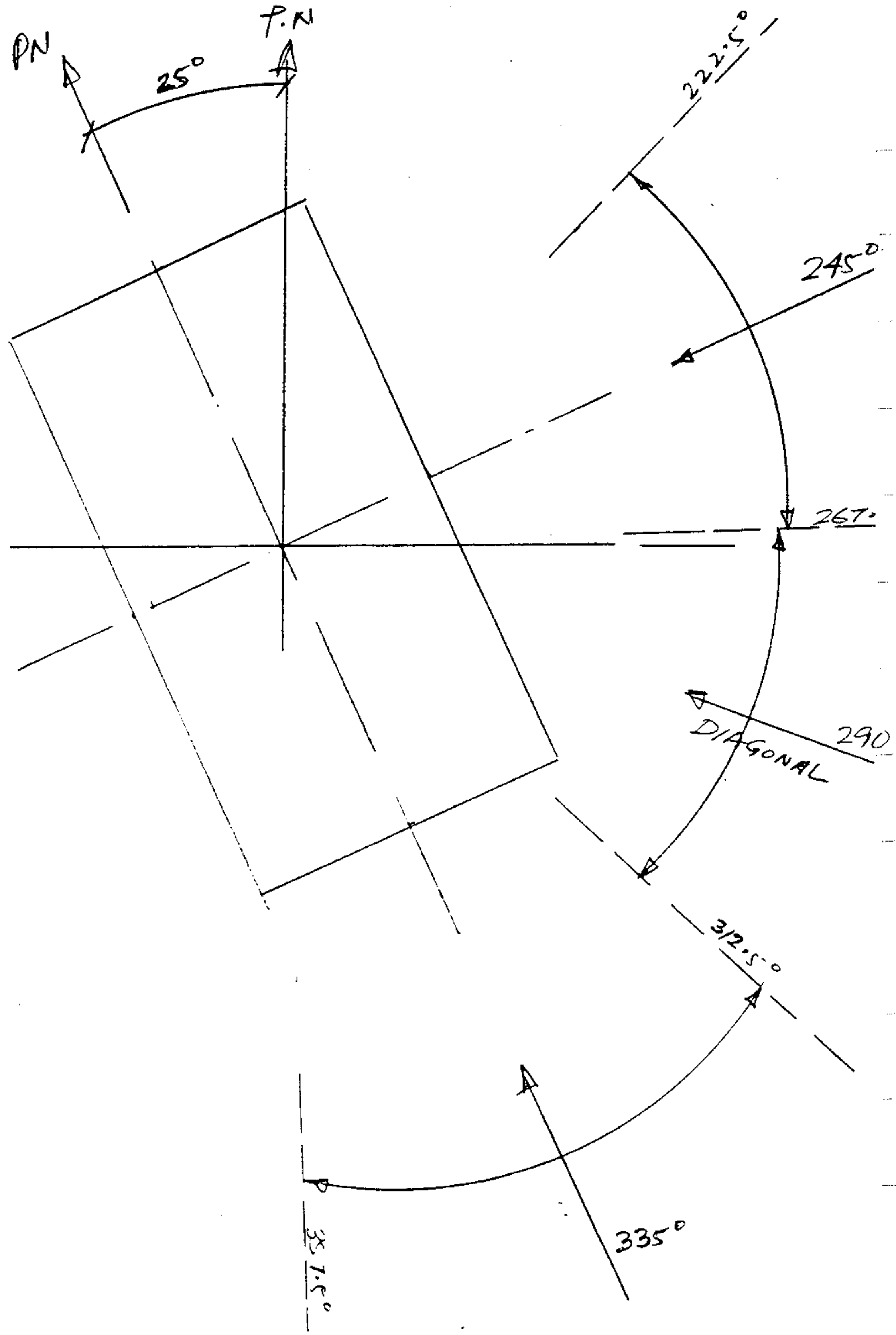
Platform ST130A

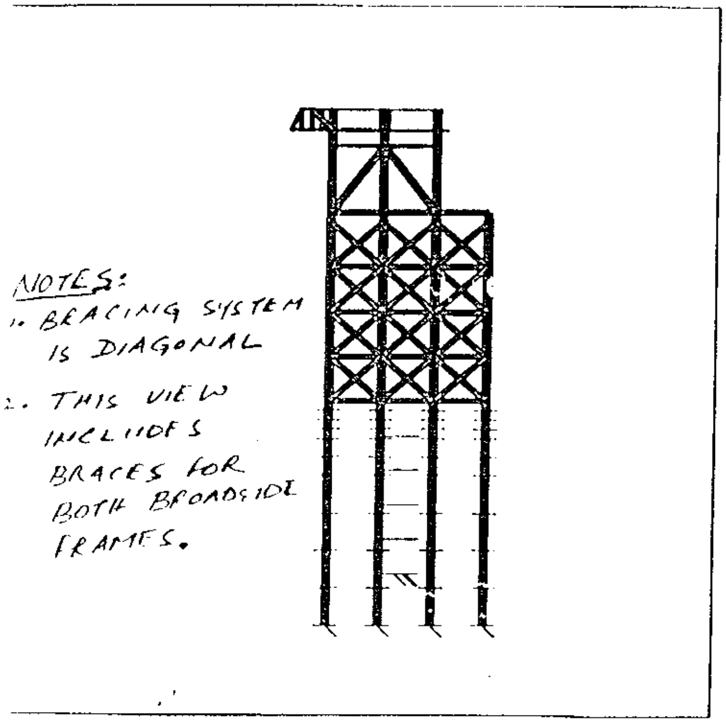
PMB ENGINEERING

By RKA Date / / Checked by Sheet No.

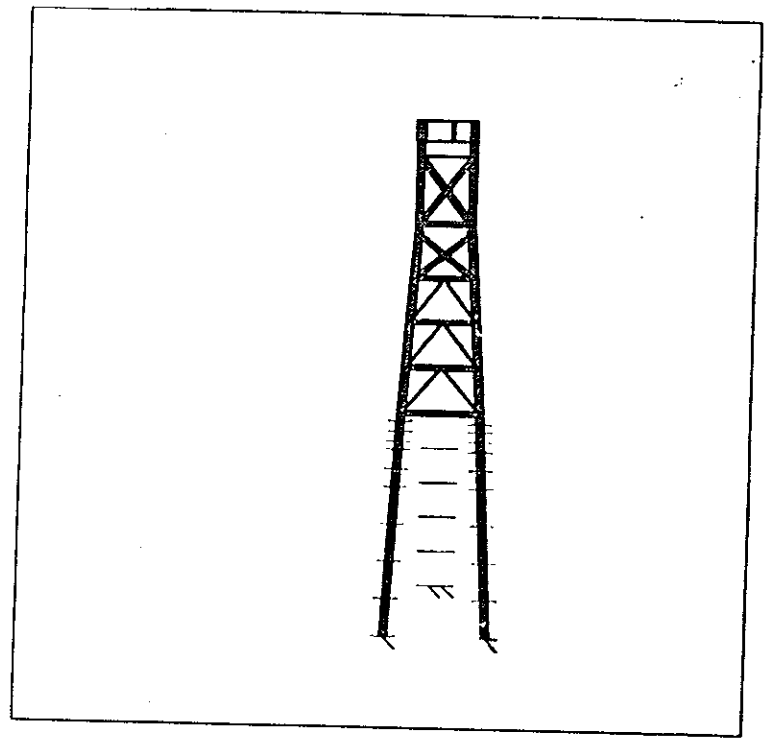
Project ANDREW JH Job No. 295

Subject ST 130 A





CAP Broadside View
Project: st130a Model: broadside Version: 1

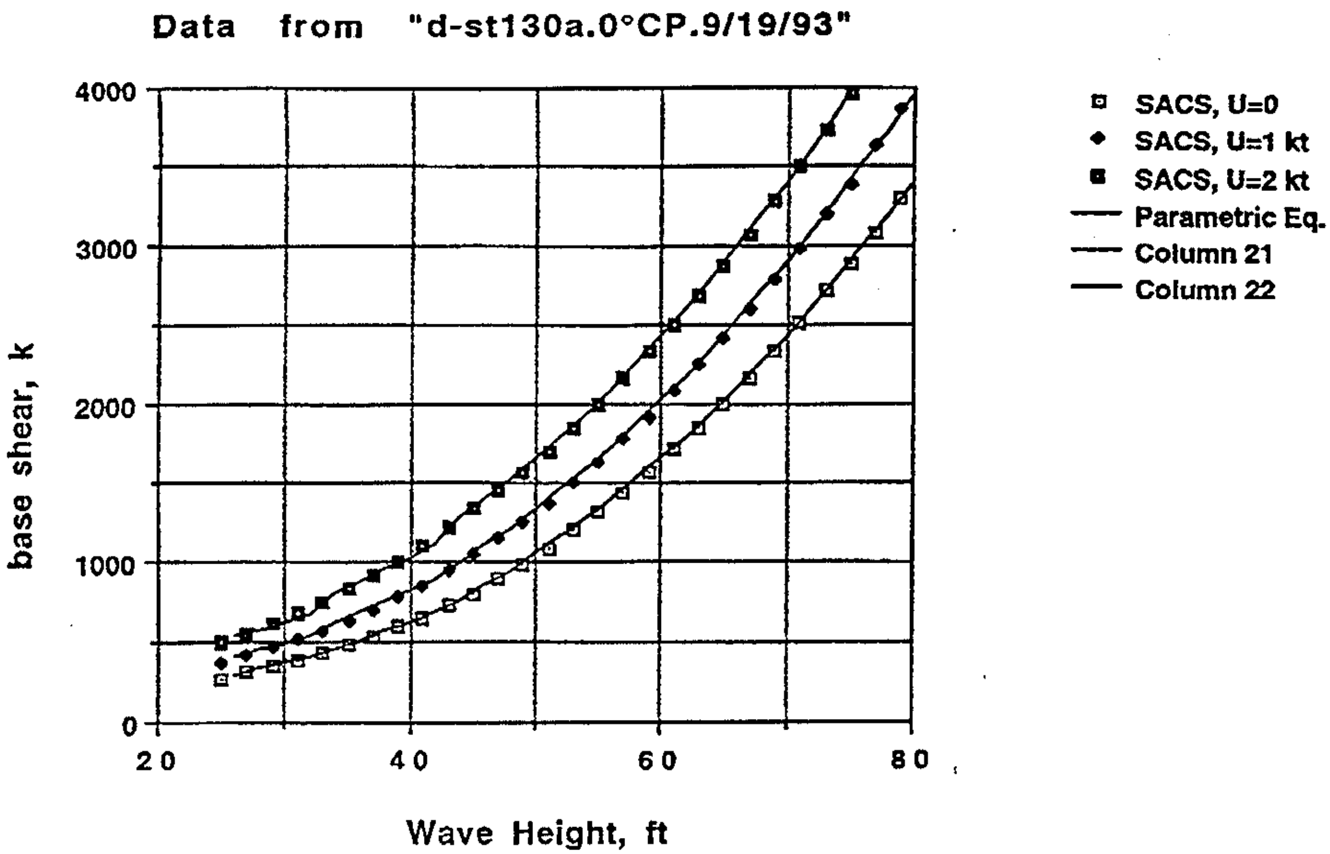


CAP End-on View
Project: st130a Model: broadside Version: 1

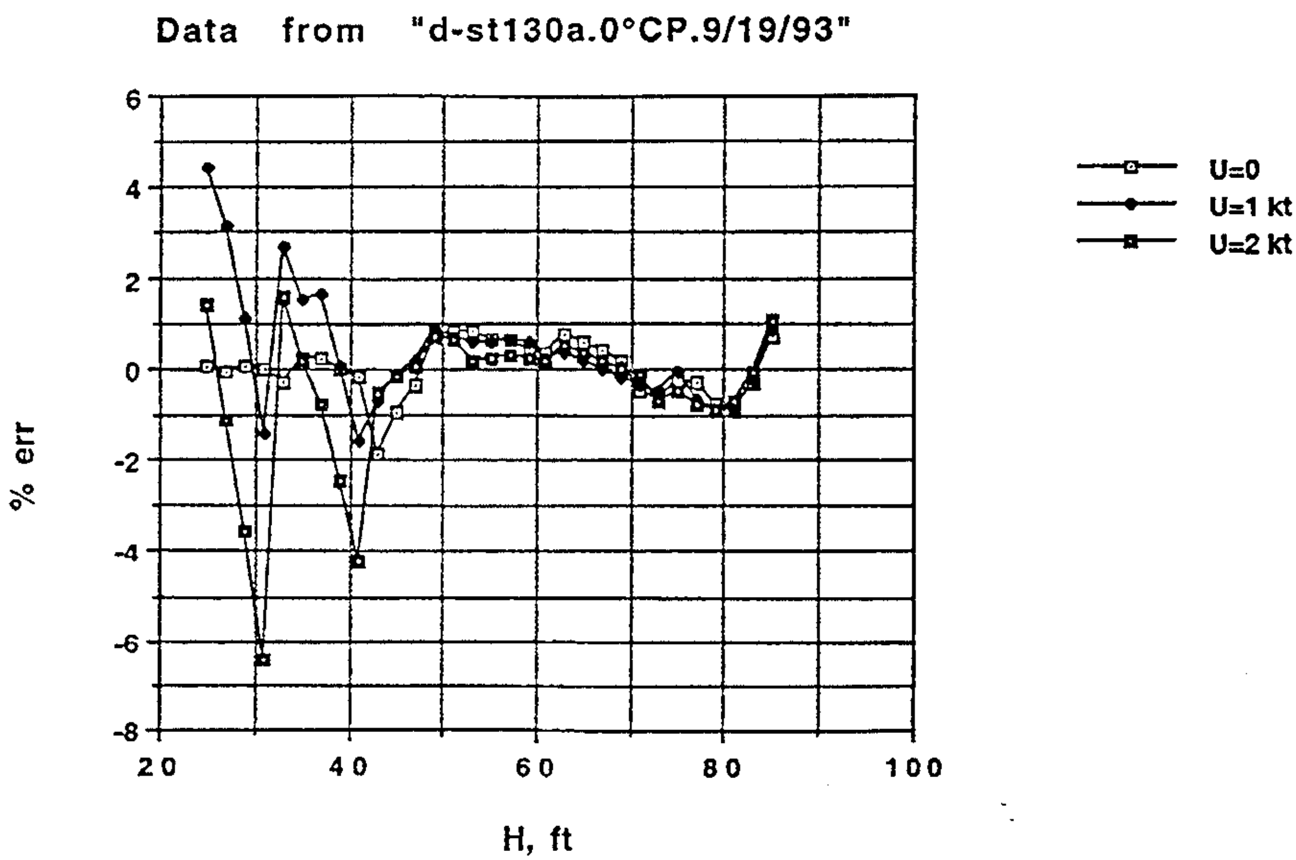
PLATFORM ST130A

C1C2C3 for ST130A(NO WIND)NEW ? Mon, Sep 20, 1993 3:01 PM

BRANCH	C1	C2	C3
Wave Direction = 0°	END-ON		
H <= 32 ft	1.922e+0	6.122e+0	1.541e+0
32 ft < H <= 42 ft	4.545e-1	5.770e+0	1.958e+0
H > 42 ft	5.725e-2	4.999e+0	2.508e+0
Wave Direction = 45°	DIAGONAL		
H <= 32 ft	4.265e+0	7.752e+0	1.354e+0
32 ft < H <= 42 ft	1.097e+0	7.198e+0	1.747e+0
H > 42 ft	8.411e-2	5.845e+0	2.427e+0
Wave Direction = 90°	BROADSIDE		
H <= 32 ft	7.571e+0	7.281e+0	1.245e+0
32 ft < H <= 42 ft	1.740e+0	6.705e+0	1.667e+0
H > 42 ft	1.309e-1	5.502e+0	2.350e+0

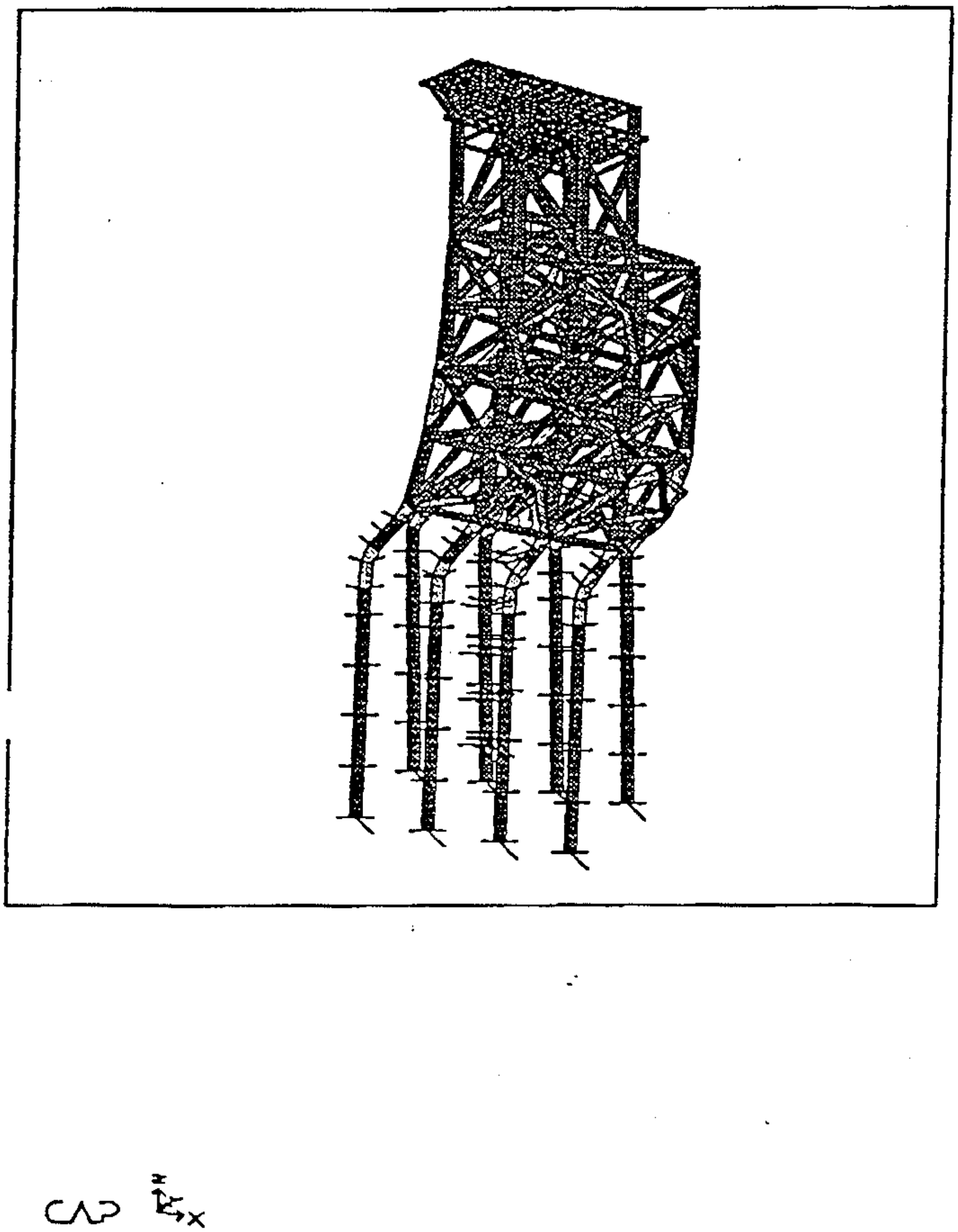


Data from "d-st130a.0°CP.9/19/93"
4000
3000
2000
1000
0
20
40
60
80
base shear, k
Wave Height, ft
SACS, U=0
SACS, U=1 kt
SACS, U=2 kt
Parametric Eq.
Column 21
Column 22



Data from "d-st130a.0°CP.9/19/93"
6
4
2
0
-2
-4
-6
-8
20
40
60
80
100
% err
H, ft
U=0
U=1 kt
U=2 kt

PLATFORM ST130A					This Platform Collapsed During Hurricane Andrew Loads								
Water Depth =		140	ft.										
Storm	Wave	Hs	H max	Peak	Zero Crossing	C1	C2	C3	U	H+C2*U	BS	Remarks	R median
Hour	Direction		1.684	Spectral T	Period								
			* Hs	Tp	Tz= 0.74* Tp					U in knots			
	(Degree)	(ft.)	(ft.)	(sec.)	(sec.)				(knots)		(Kips)		(Kips)
Broadside Direction:			(222.5 deg. to 267.5 deg.)										2450.00
1	252.0	18.19	30.618	9.957	7.37	7.571	7.281	1.245	0.59	34.923	631.43		
2	253.5	20.87	35.136	10.901	8.07	1.74	6.705	1.667	0.74	40.112	818.87		
3	253.1	23.28	39.185	11.275	8.34	1.74	6.705	1.667	0.94	45.512	1010.74		
4	255.2	26.06	43.880	11.922	8.82	0.1309	5.502	2.35	1.20	50.489	1316.57		
5	258.6	29.76	50.099	12.742	9.43	0.1309	5.502	2.35	1.52	58.457	1857.82		
Diagonal Direction:			(267.5 deg. to 312.5 deg.)										3000.00
6	268.5	33.98	57.213	13.674	10.12	0.08411	5.845	2.427	1.85	68.022	2358.80		
7	286.6	36.21	60.952	14.427	10.68	0.08411	5.845	2.427	2.02	72.773	2778.72		
8	305.8	33.50	56.390	13.829	10.23	0.08411	5.845	2.427	1.75	66.610	2241.72		
End-On Direction:			(312.5 deg. to 357.5deg.)										2800.00
9	314.1	31.11	52.369	12.928	9.57	0.05725	4.999	2.508	1.33	59.012	1582.33		
10	317.7	29.02	48.851	12.234	9.05	0.05725	4.999	2.508	0.91	53.377	1230.19		
11	321.5	27.54	46.363	11.764	8.70	0.05725	4.999	2.508	0.51	48.916	988.38		
12	324.7	25.82	43.474	11.394	8.43	0.05725	4.999	2.508	0.29	44.945	799.30		
13	327.3	24.04	40.464	10.920	8.08	0.4545	5.77	1.958	0.10	41.062	655.62		
14	329.1	22.56	37.981	10.673	7.90	0.4545	5.77	1.958	0.01	38.016	563.78		
15	330.7	21.23	35.736	10.518	7.78	0.4545	5.77	1.958	0.00	35.736	499.47		
16	331.6	20.00	33.670	10.251	7.59	0.4545	5.77	1.958	0.00	33.670	444.50		
17	331.6	18.69	31.462	9.875	7.31	0.4545	5.77	1.958	0.00	31.462	389.22		



ST130A DEFORMED SHAPE AND NONLINEAR PILES AND BRACES AT LOAD STEP #10 AT FAILURE FOR THE DIAGONAL LOAD CASE

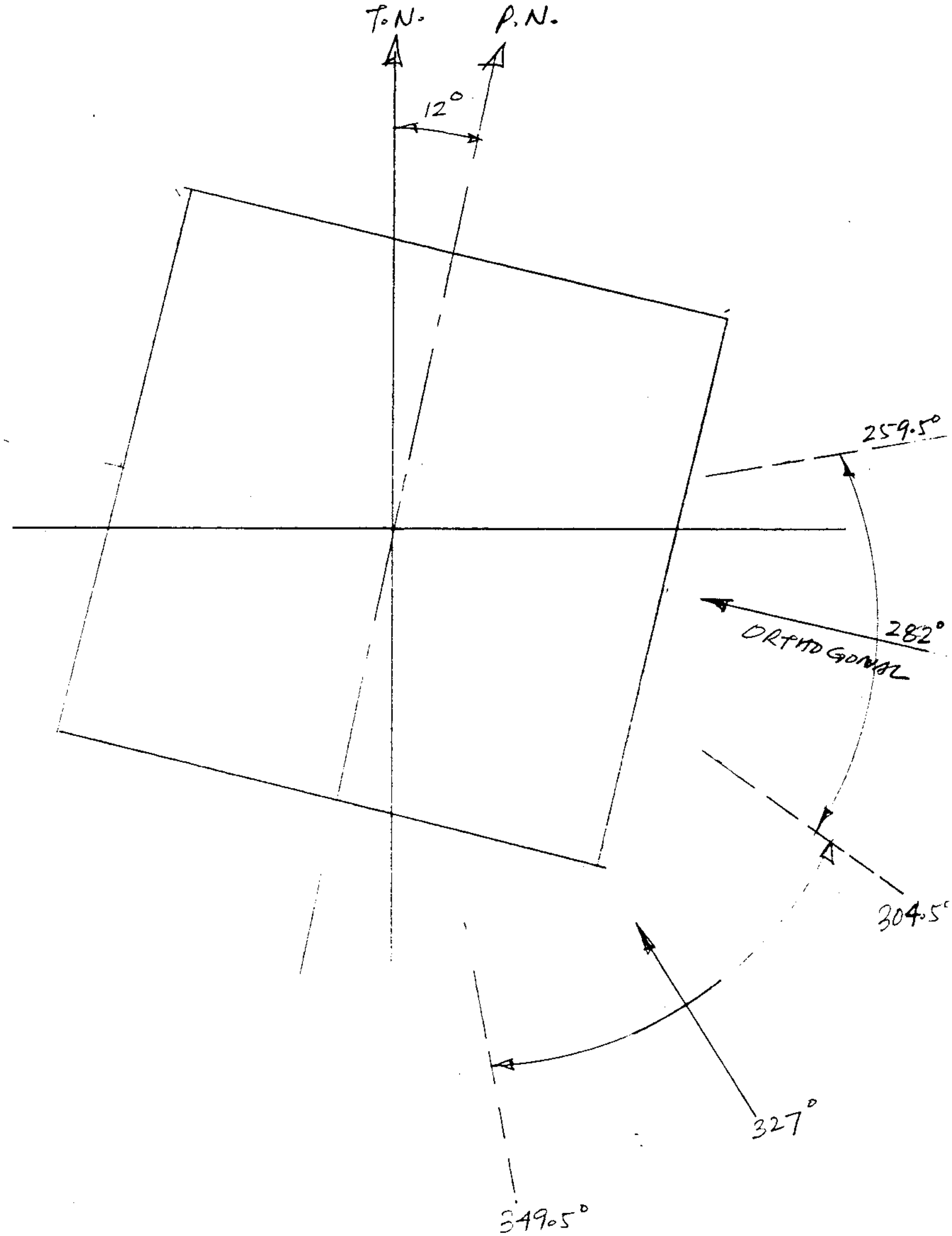
Platform T21

PMB ENGINEERING

By: RKA Date: 06/07/93 Checked by: Sheet No.:

Project: ANDREW JIP Job No.: 295

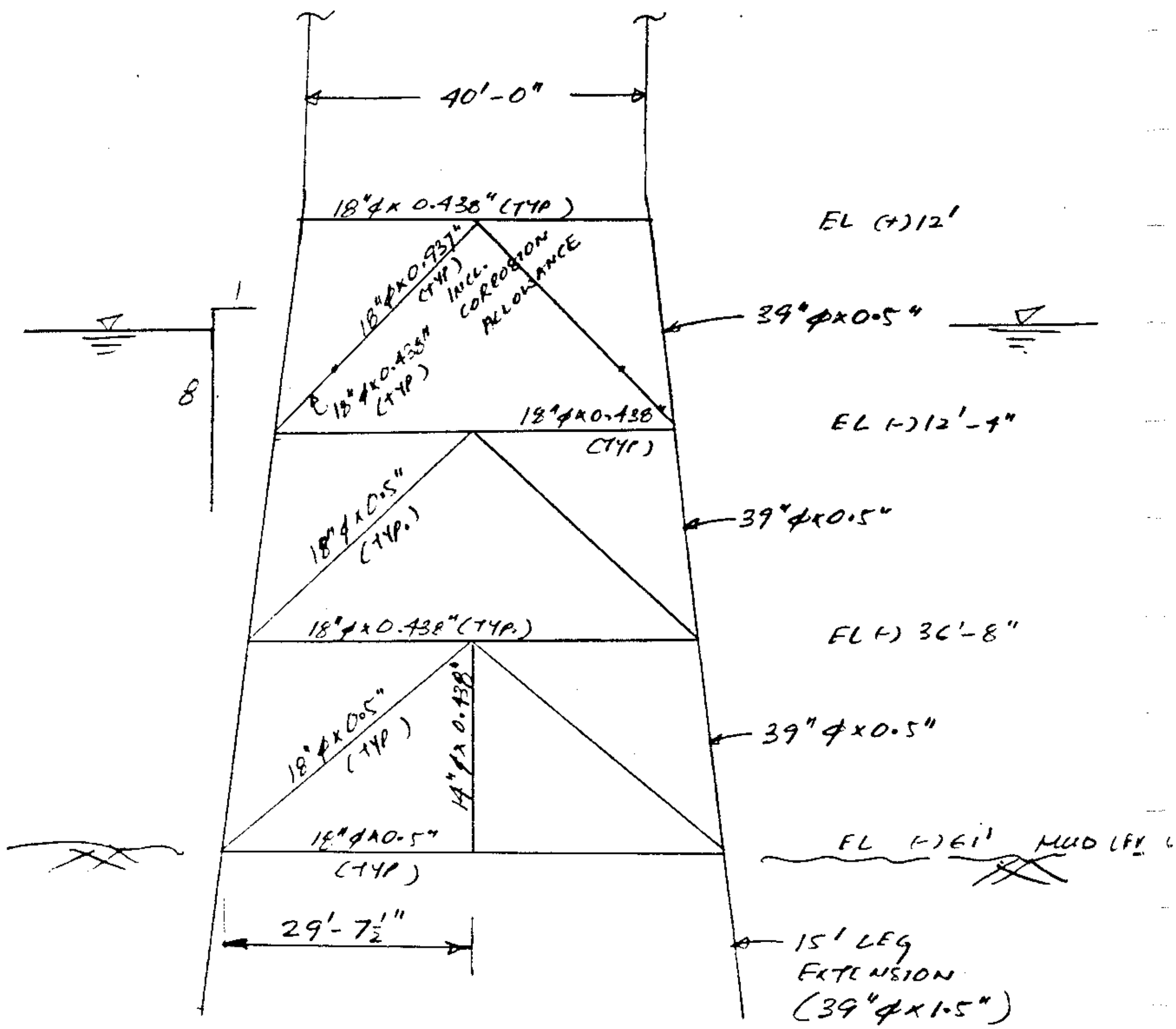
Subject: PLATFORM ST 72 T-21



By________ Date / / Checked by________ Sheet No.________

Project ANDREW JIP Job No. 295

Subject PLATFORM T-21 (ST-72)



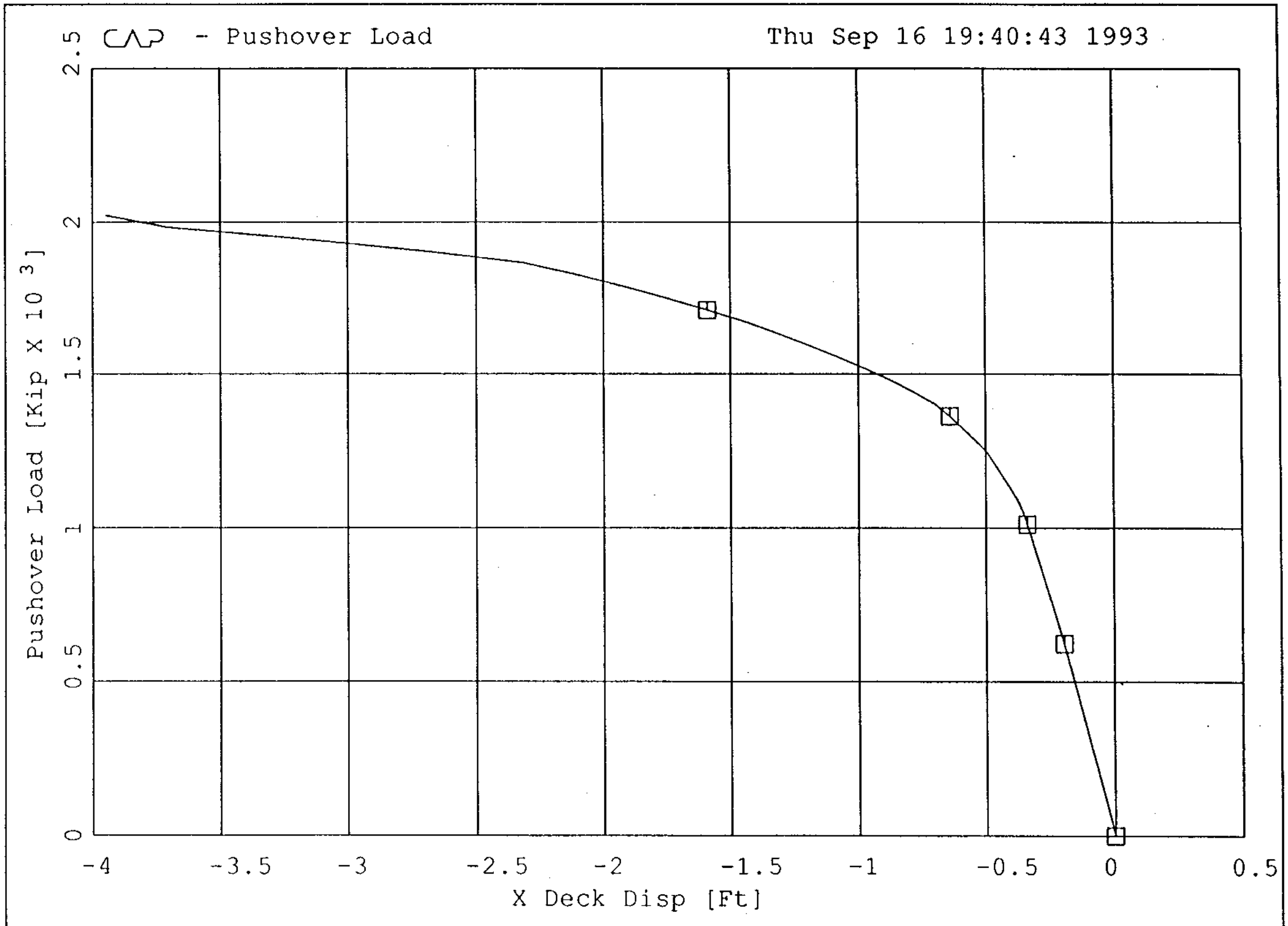
TYPICAL ELEVATION (PLATFORM T-21)

PLATFORM ST72 (T21) — This Platform Collapsed Against Hurricane Andrew Loads

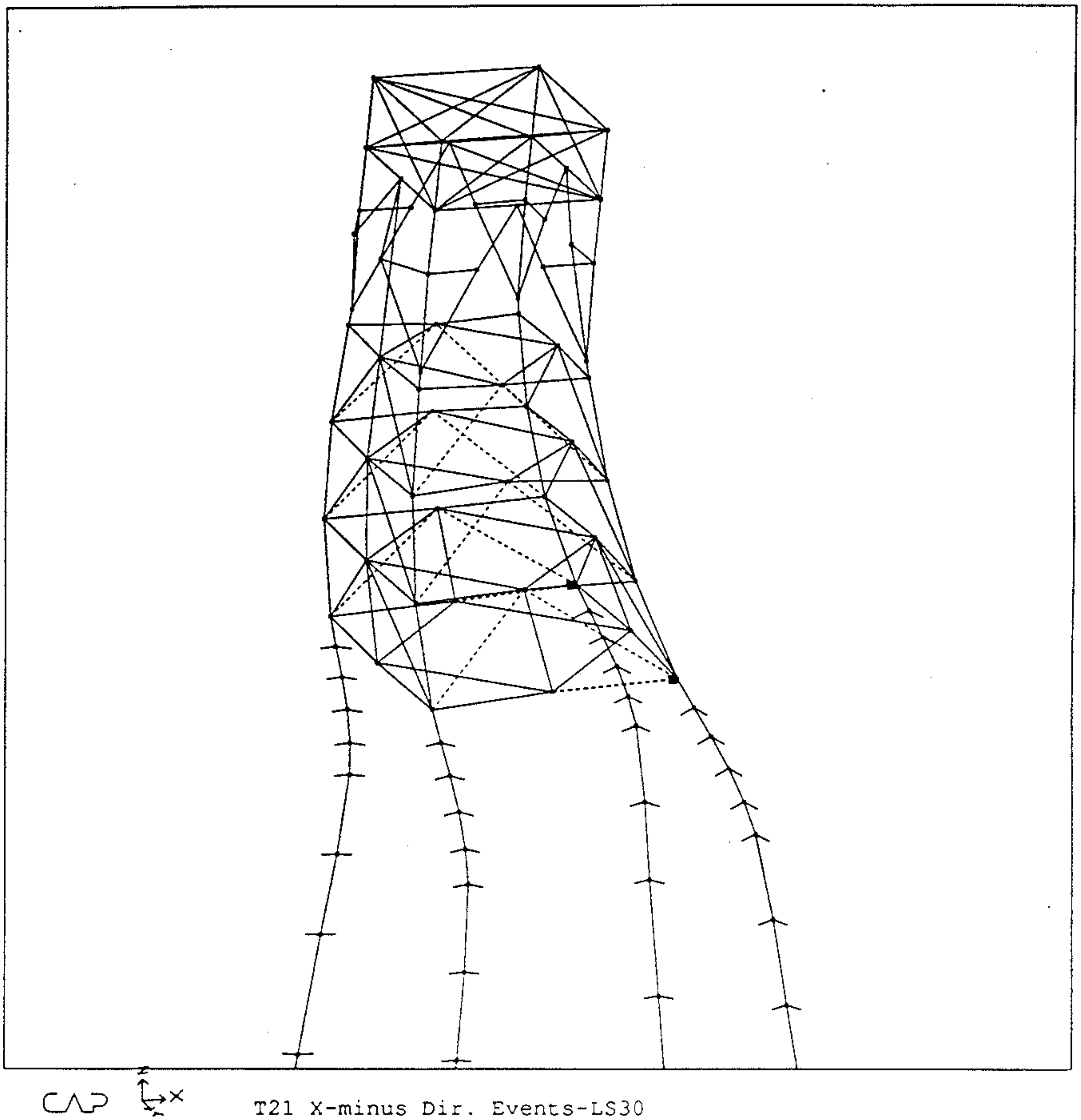
Water Depth = 61.000 ft.

Storm Hour	Wave Direction (Degree)	Hs (ft.)	H max 1.711 *Hs (ft.)	Peak Spectral T Tp (sec.)	Zero Crossin Period Tz= 0.74* Tp (sec.)	C1	C2	C3	U (ft/sec)	U Platform (ft/sec)	H+C2*U U in ft/sec	BS (Kips)	Remarks	R median (Kips)
Diagonal_1 Direction			**(less than 259.5 deg.)**											
1	247.79	17.44	29.84	14.33	10.61	0.892015	2.30524	1.84404	1.93	1.64	33.618	582.69		
2	247.74	19.30	33.02	14.36	10.63	0.892015	2.30524	1.84404	2.27	1.93	37.472	711.78		
3	242.82	21.48	36.74	11.04	8.17	0.892015	2.30524	1.84404	2.71	2.31	42.058	880.68		
4	238.21	24.97	42.72	11.91	8.82	0.892015	2.30524	1.84404	3.29	2.80	49.163	1174.43		
Orthogonal Direction			**(259.5 deg. to 304.5 deg.)**											1984.00
5	257.31	29.07	49.74	13.17	9.74	1.11731	2.56927	1.78824	4.26	3.41	58.501	1615.42		
6	291.42	29.84	47.58	15.71	11.63	1.11731	2.56927	1.78824	4.26	3.41	56.342	1510.35		
Diagonal_2 Direction			**(304.5 deg. to 349.5 deg.)**											
7	314.01	27.74	47.46	14.45	10.69	0.892015	2.30524	1.84404	3.17	2.69	53.668	1380.50		
8	326.93	26.16	44.76	13.20	9.77	0.892015	2.30524	1.84404	2.51	2.13	49.674	1197.04		
9	333.11	25.30	43.28	11.99	8.87	0.892015	2.30524	1.84404	2.07	1.76	47.339	1095.34		
10	340.49	23.49	40.18	11.65	8.62	0.892015	2.30524	1.84404	1.33	1.13	42.790	909.13		
11	342.56	22.31	38.16	11.15	8.25	0.892015	2.30524	1.84404	0.70	0.60	39.540	785.90		
12	341.75	20.87	35.71	10.80	7.99	0.892015	2.30524	1.84404	0.26	0.22	36.220	668.55		
13	342.06	19.31	33.03	10.45	7.73	0.892015	2.30524	1.84404	0.17	0.14	33.356	574.35		
14	341.98	17.90	30.63	10.04	7.43	0.033507	1.62138	2.81546	0.44	0.37	31.232	540.88		
15	339.54	16.81	28.76	9.76	7.22	0.033507	1.62138	2.81546	0.47	0.40	29.409	456.67		
16	339.20	15.77	26.97	9.35	6.92	0.033507	1.62138	2.81546	0.47	0.40	27.625	382.89		

Project: T21 Model: X-minus Version: 2



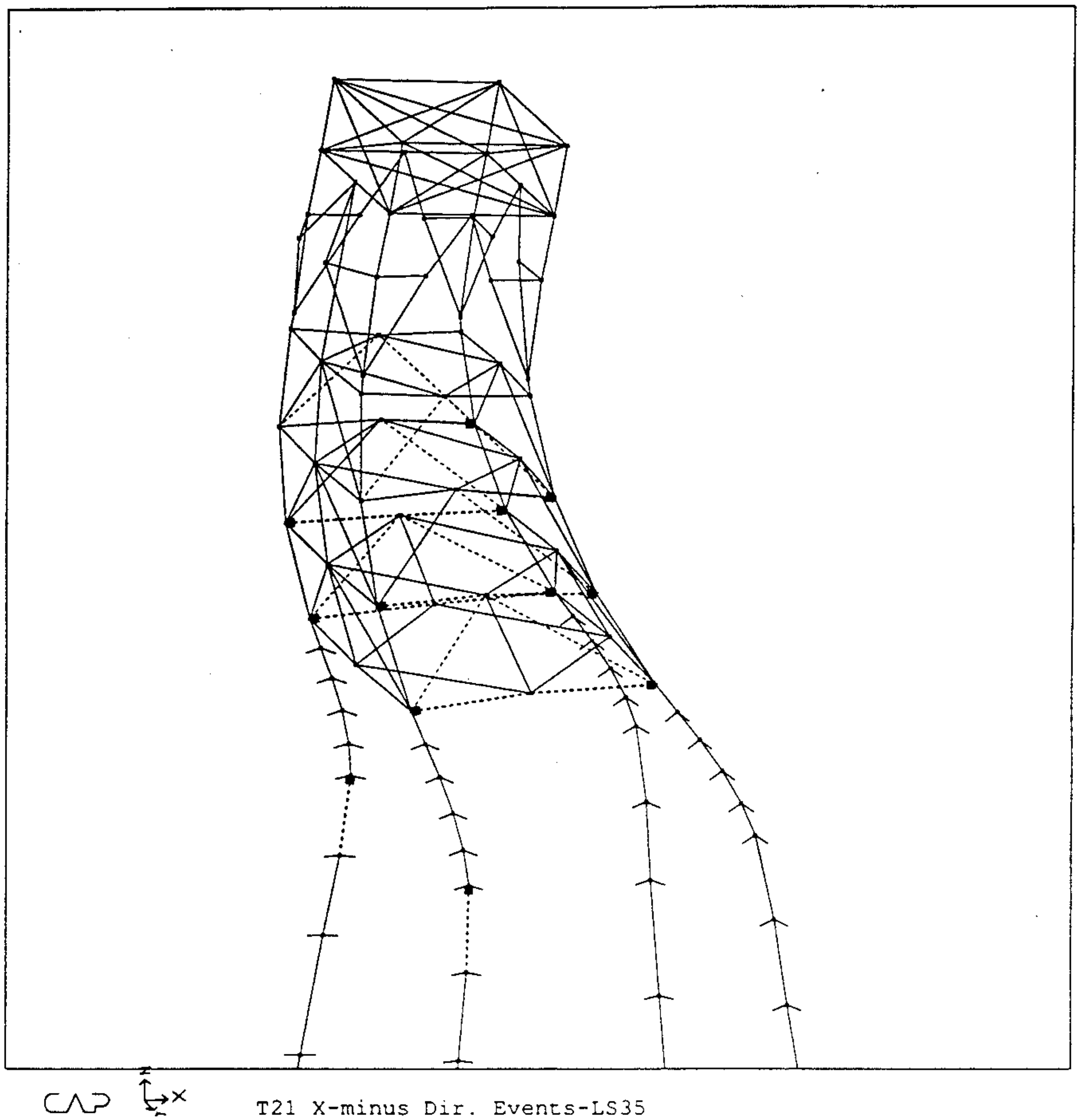
T21 X-minus



CAP

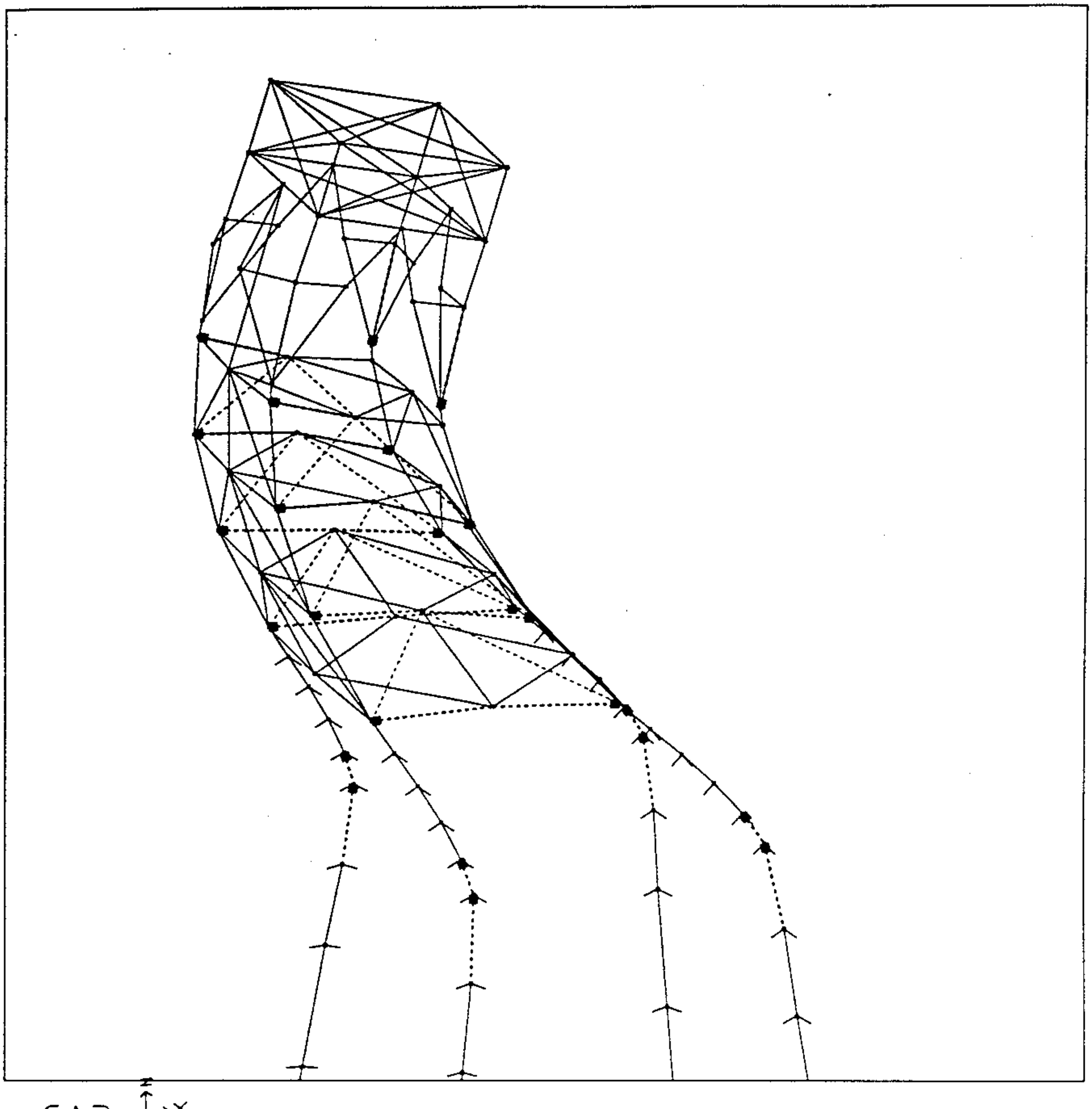
T21 X-minus Dir. Events-LS30

Project: T21 Model: X-minus Version: 2



T21 X-minus Dir. Events-LS35

Project: T21 Model: X-minus Version: 2



CAP

T21 X-minus Dir. Events-LS40

Project: T21 Model: X-minus Version: 2

Appendix C
Sensitivity Studies

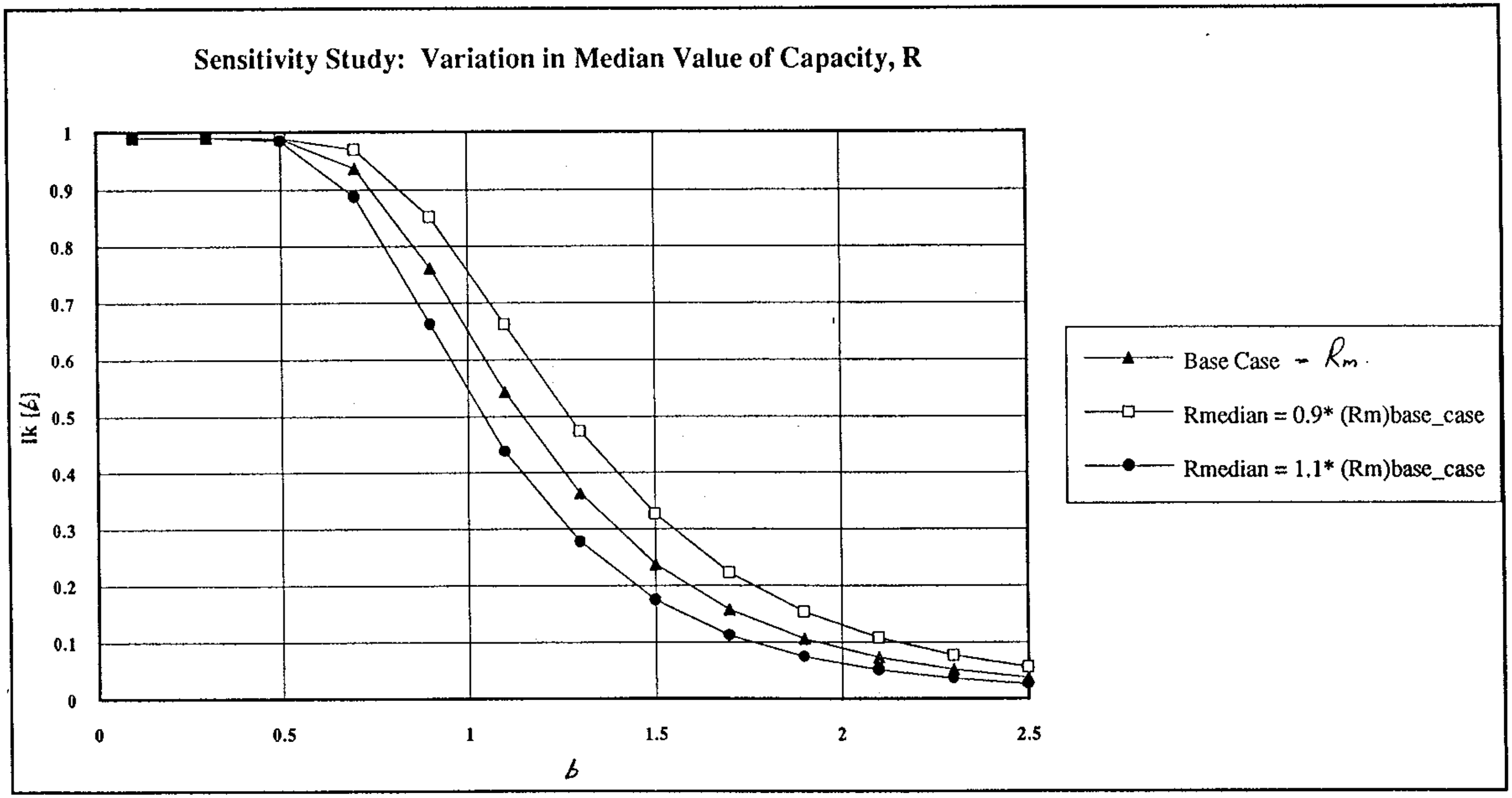

Sensitivity Study: Variation in Median Value of Capacity, R
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case = Rm
Rmedian = 0.9* (Rm)base_case
Rmedian = 1.1* (Rm)base_case

FIG. C-1

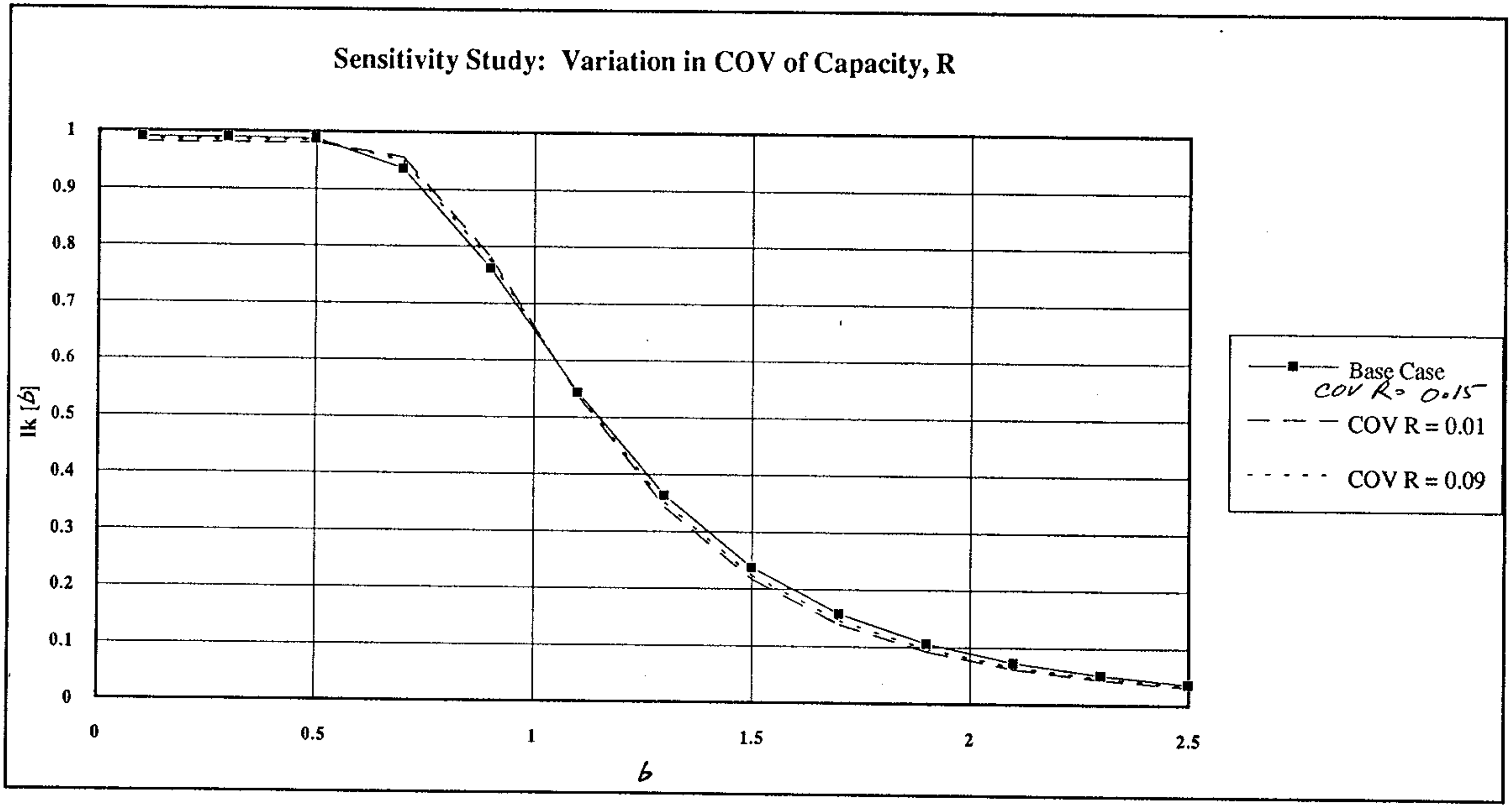

Sensitivity Study: Variation in COV of Capacity, R
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case
COV R= 0.15
COV R = 0.01
COV R = 0.09

FIG. C-2

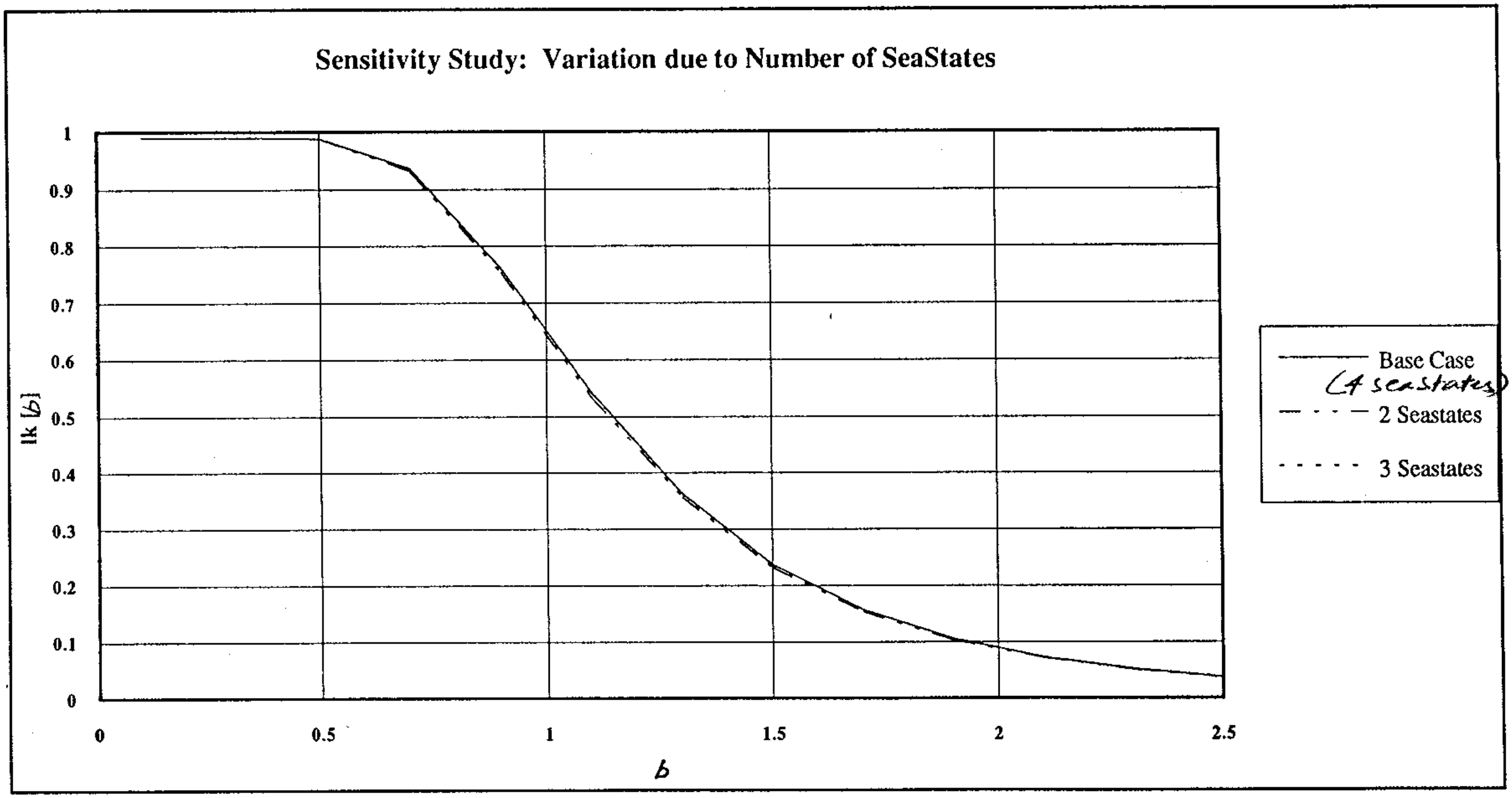

Sensitivity Study: Variation due to Number of SeaStates
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case
(4 seastates)
2 Seastates
3 Seastates

FIG. C-3

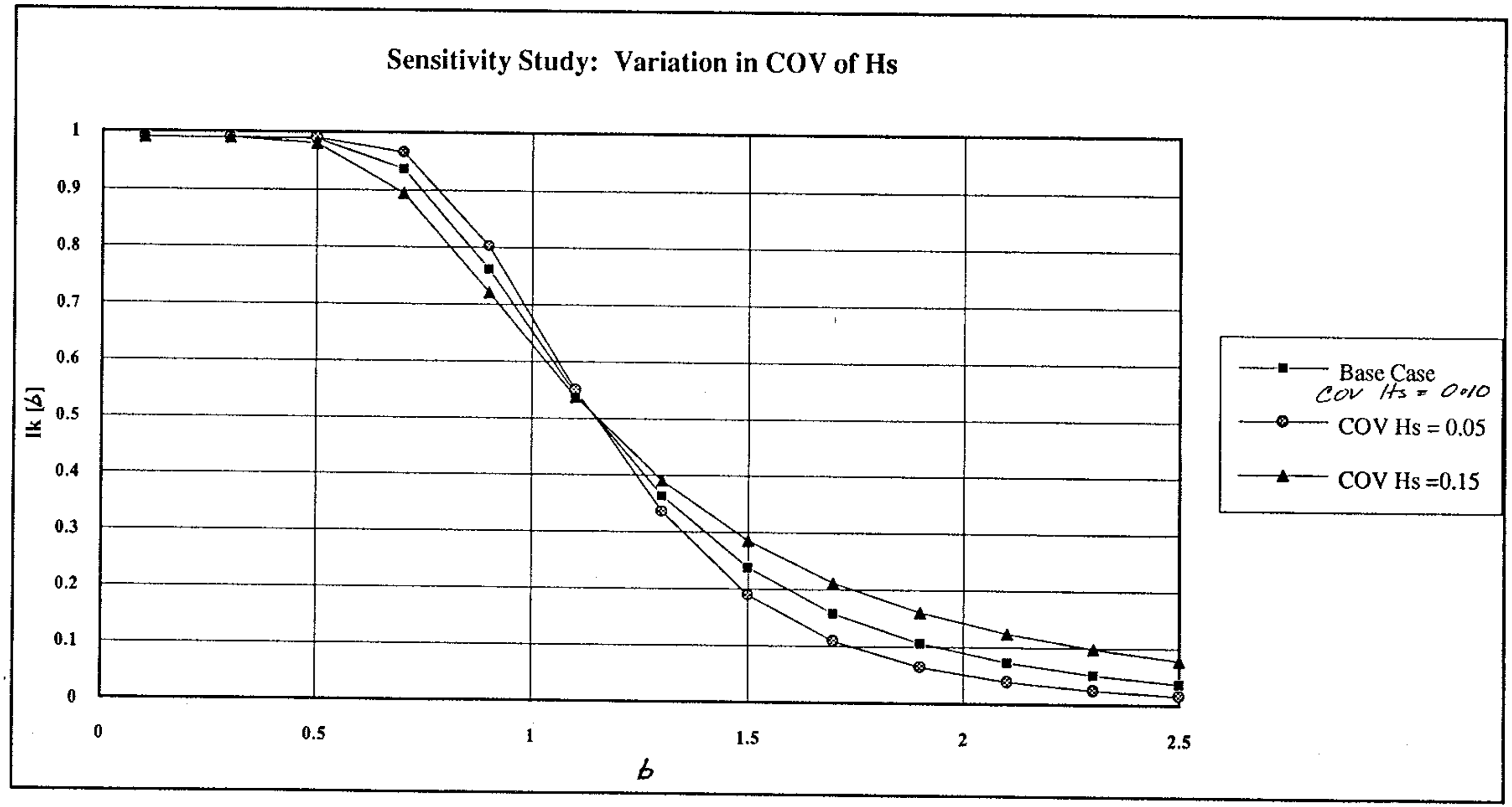

Sensitivity Study: Variation in COV of Hs
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case
COV Hs = 0.10
COV Hs = 0.05
COV Hs =0.15

FIG. C-4

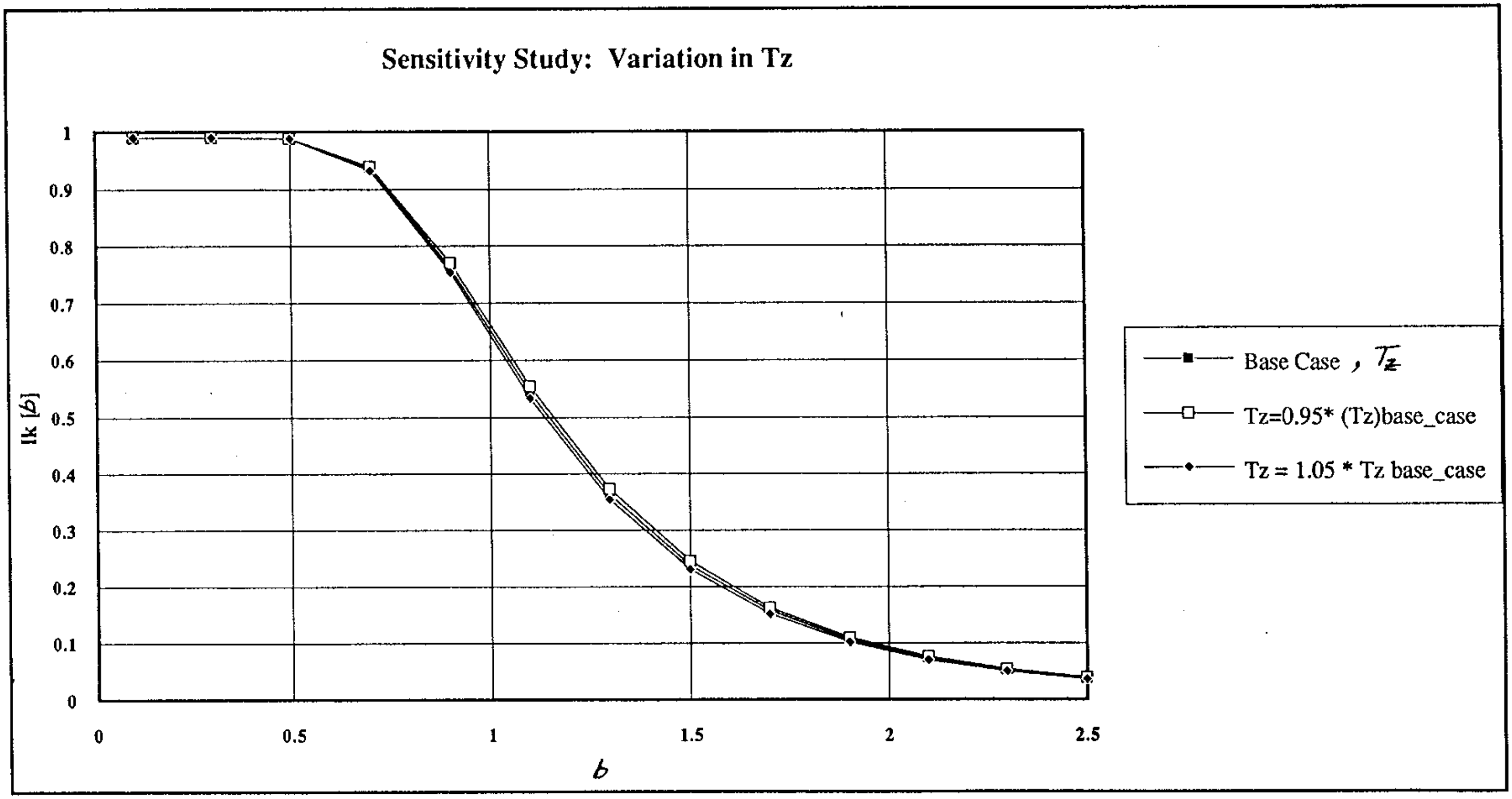

Sensitivity Study: Variation in Tz
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case , Tz
Tz=0.95* (Tz)base_case
Tz = 1.05 * Tz base_case

FIG. C-5

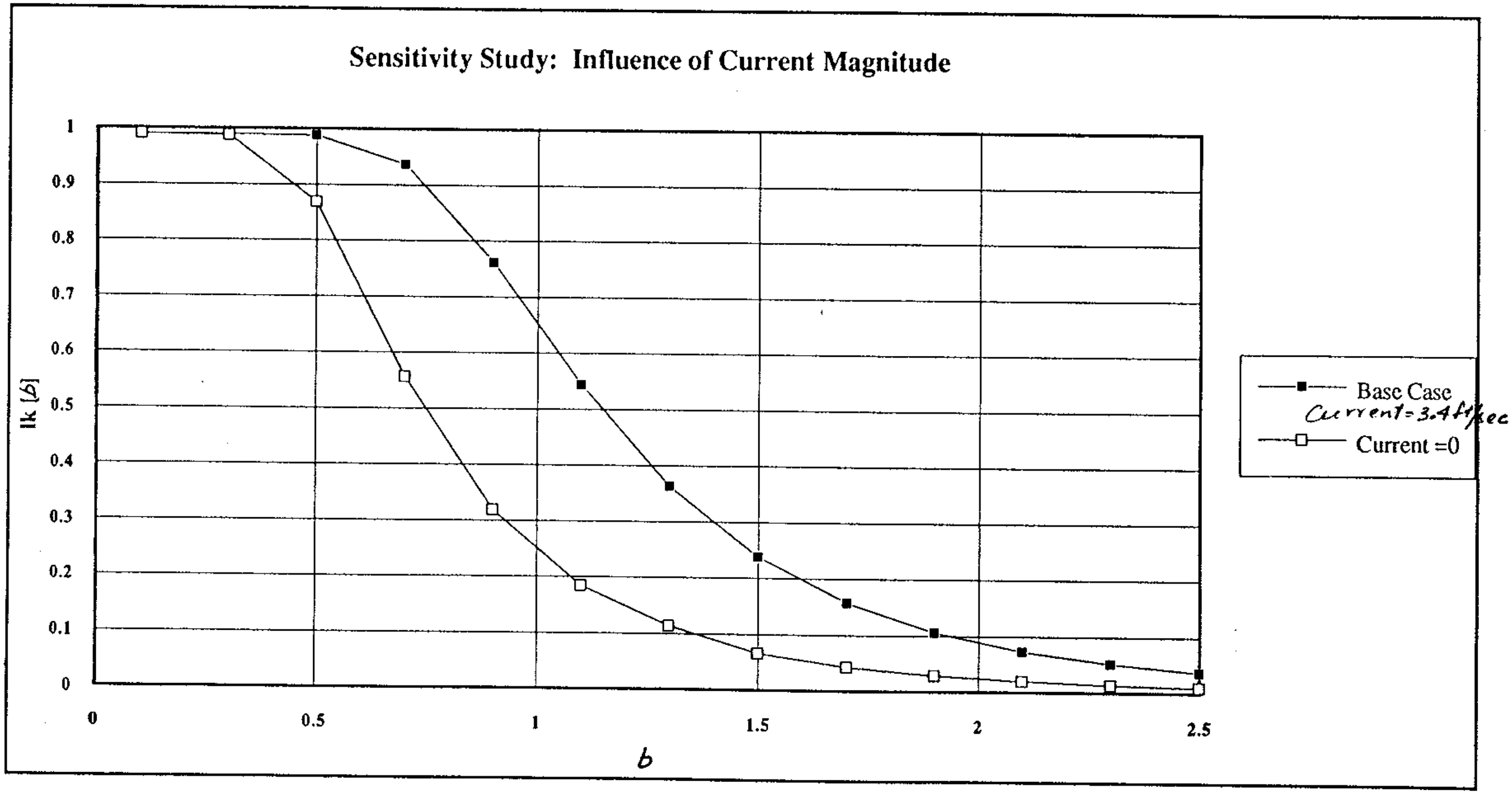

Sensitivity Study: Influence of Current Magnitude
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case
Current=3.4ft/sec.
Current =0

FIG. C-6

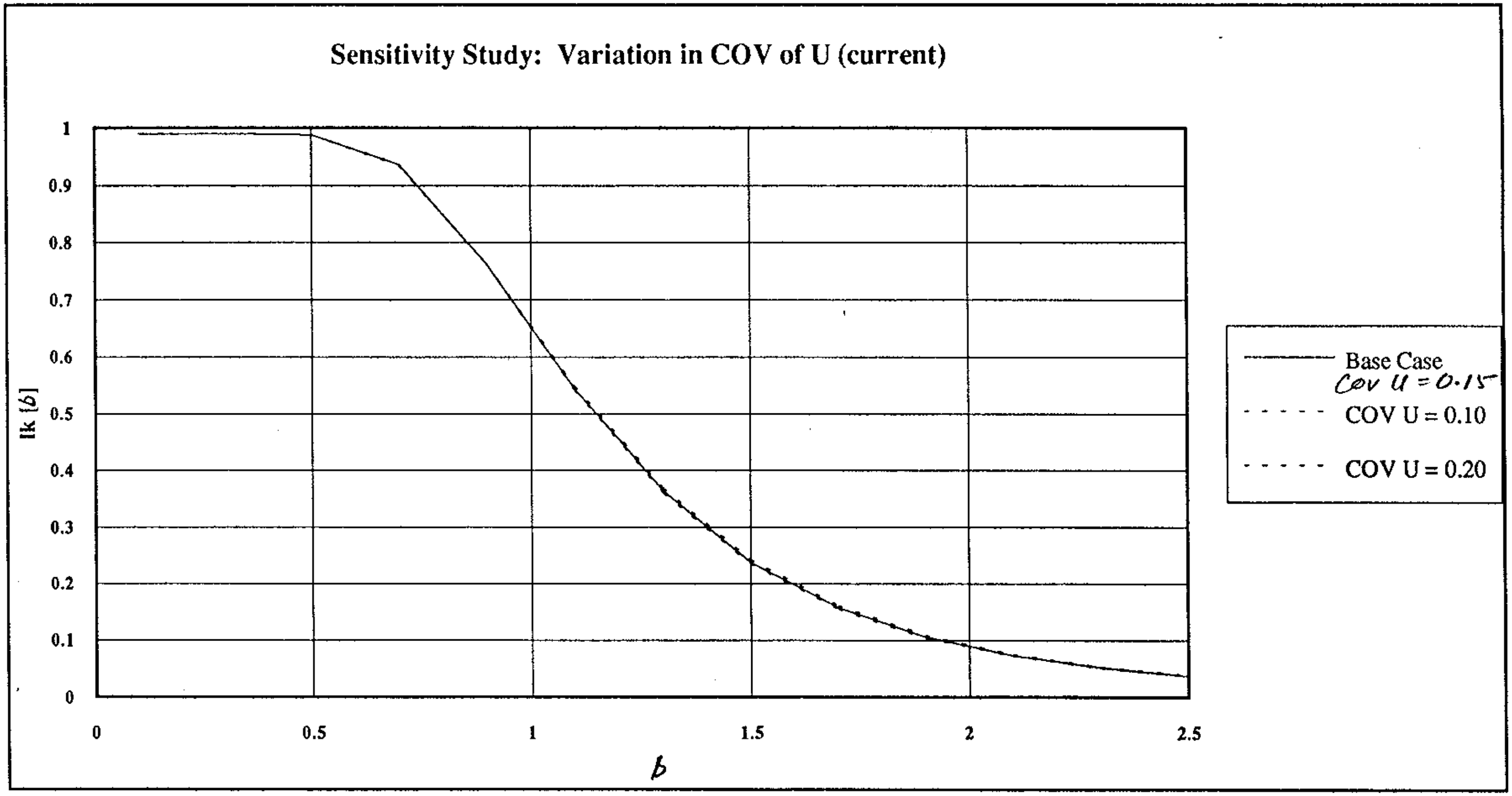

Sensitivity Study: Variation in COV of U (current)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Ik [b]
0
0.5
1
1.5
2
2.5
b
Base Case
Cov U = 0.15
COV U = 0.10
COV U = 0.20

FIG. C-7

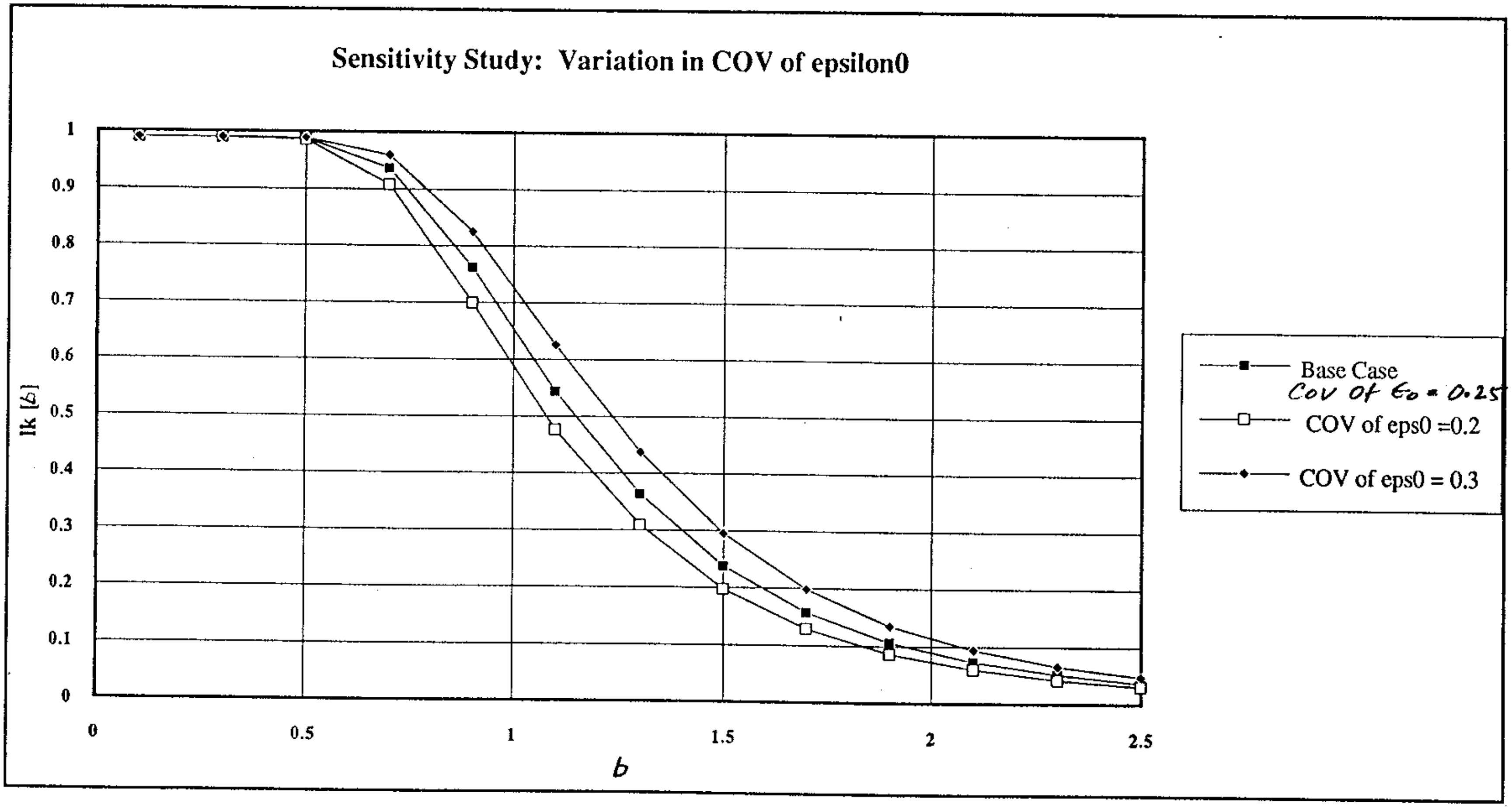

Sensitivity Study: Variation in COV of epsilon0
Ik [b]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
b
Base Case
COV of ε0 = 0.25
COV of eps0 =0.2
COV of eps0 = 0.3

FIG. C-8

Appendix D
Gulf of Mexico Versus North Sea Storms

A comparative discussion of peak sea states generated in tropical cyclones and extratropical cyclones.

The maximum sea state hindcast in hurricane Andrew in the Gulf of Mexico was approximately 13 meters in significant wave height (HS) with associated peak spectral period (TP) of 14 seconds and maximum individual wave height of 22 meters. This is among the most extreme sea states either hindcast or measured in intense tropical cyclones (TC). In this century, we estimate that only 5 other hurricanes attained central pressure of 932 mb or lower in the Gulf of Mexico. If we use the ODGP historical hindcasts as a reference the maximum hindcast HS at the shelf break south of the Mississippi Delta of 13.2 m (43.5 ft) was exceeded in only 7 storms this century anywhere in the Gulf. Off the central Gulf Coast, only Betsy (1965) with central minimum pressure of 948 mb, and Camille (1969) with minimum central pressure of 908 mb, both with peak hindcast HS of 14 meters exceeded the peak hindcast HS in Andrew. The maximum measured HS in Camille at a site near the eye was 13.6 m with associated TP of 13.1 seconds. More recently, a NOAA data buoy in the eastern Gulf of Mexico measured a peak HS of 10.7 m with TP of 12.5 seconds in Hurricane Kate (1985) with minimum central pressure of 956 mb. The maximum wind speed measured in Kate from the same buoy was 47.4 m/sec (8.5 minute average at 10 m height), a record at the time for NOAA buoys moored in US waters.

From the above evidence we may conclude tentatively that in the most intense tropical cyclones (TC) of recent record (e.g. category 4 and 5 hurricanes in the Atlantic basin, supertyphoons in the western North Pacific basin) the most severe sea states are in the range of HS of 13-15 m with TP in the narrow range of 12-14 seconds and associated maximum wind speeds (30-minute average at 20 m height) of about 50 m/sec +/- 5 m/sec. Hindcast models and a few measurements only suggest the shape of the directional spectra of these waves and indicate in general that in the inner core of intense TC, the waves possess a rather broad spread of wave energy often with bimodal directional maxima in wave energy, except that the directional spectrum simplifies to a narrow unimodal distribution only to the right (looking down the storm track) of the storm center just outside the eye wall (or radial wind speed maximum), which is also usually the locus of the maximum sea state in the storm.

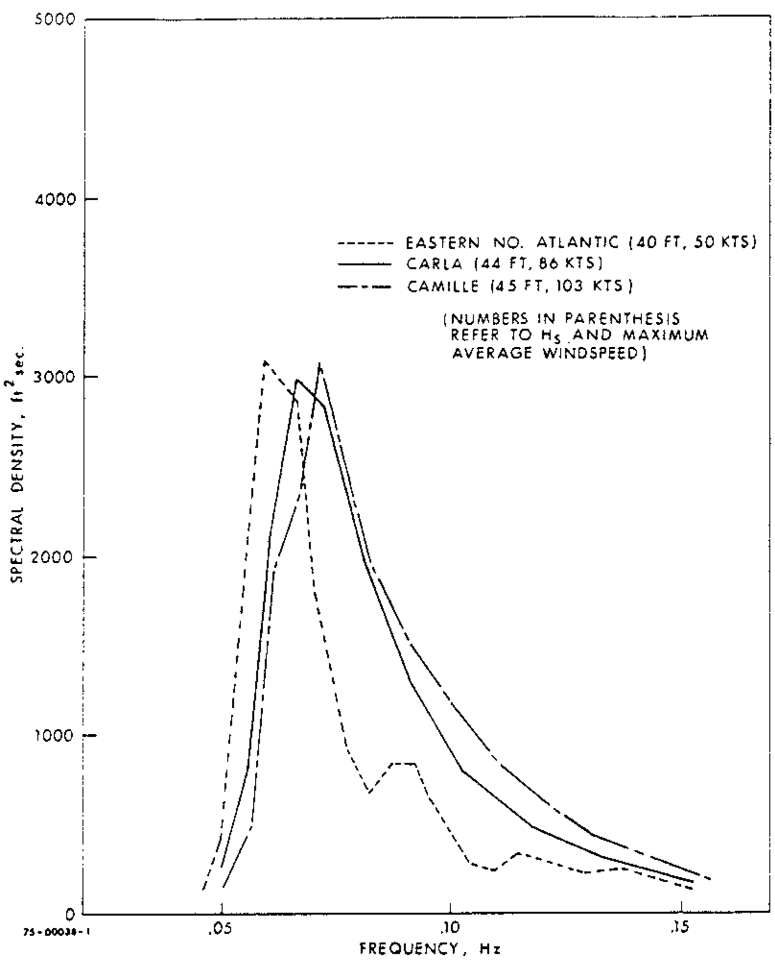
How do these sea states compare with those generated by the most intense extratropical cyclones (ETC)? Certainly sea states (in terms of HS) comparable to those quoted above have been observed almost routinely in typically intense open ocean ETC. For example, Figure 1 compares the hindcast spectra in hurricanes with those measured in an eastern North Atlantic ETC, for HS in the range of 12-14 meters. Note that while the total wave energy is comparable between these three sea states, there is a systematic shift in the peak of the spectrum to the left toward lower frequency as the scale of the storm increases from the very tight structure of Camille (radius of maximum wind of about 10 n.

mi.) to the broader structure of hurricane Carla (radius of maximum wind of about 25 n. mi.) to the typically broad structure of the wind field in ETC (radius of maximum wind of tens to hundreds of miles). Physically, the larger scales of the wind field allows longer fetches for wave development and therefore the generation of lower frequencies in the wave spectrum. In the ETC spectrum shown the peak period of 15.4 seconds. There are differences in the high-frequency part of the spectrum where the level of energy appears to be proportional to the local wind speed rather than the variance of the spectrum. The variation of the level of the spectrum in the high frequency rear face of the spectrum (usually called the equilibrium range) is actually more closely related to the wave age or stage of wave development. TC peak sea states are typically "younger" or less well developed in terms of wave age than ETC peak sea states.

One is tempted to conclude that the significant wave steepness, HS/TP, is systematically larger in TC than ETC peak sea states. However, occasionally, TC wind fields will develop anomalous shapes characterized by very large radii of maximum winds, with maximum winds rather lower than expected for TC of a given minimum central pressure. For example, as Hurricane Gloria (1985) (minimum central pressure of 942 mb) passed over a NOAA data buoy moored in deep water of the East Coast of the US , peak HS of 14.3 meters with associated TP of 16.7 seconds were measured, while the peak wind speed measured was only 25.2 m/sec. In the 24-hour period prior to this measurement, aircraft data showed that the wind field in Gloria had changed dramatically from that typical of an intense TC (small radius of maximum wind) to that more typical of a small intense ETC.

Measurements of directional spectra of intense ETC generated sea states are rare. However, model results suggest that the directional spread of peak sea states in ETC is smaller than that of TC, with less propensity for bimodal distributions. This is related to the smaller curvatures in fetch generation zones of ETC relative to those of TC.

Finally, we should remark that in recent years the most extreme measured sea states have tended to be recorded in ETC rather than TC, though this is no guarantee that ETC have greater ultimate wave generation potential than TC. For example, there have been several extremes of HS between 15 and 16 meters measured by US and Canadian buoys moored in deep water in the Northeast Pacific Ocean during ETC. In the so-called "Halloween Storm" of October 31, 1991 a buoy moored south of Nova Scotia measured a peak HS of 17.4 meters (TP 18.3 seconds). The same buoy recorded a peak HS of 16.3 m (TP 17.1 seconds) in the so-called "Storm of the Century" of March 13, 1993. Comparable extreme sea states have been measured in some of the unusually intense ETC which have affected the Northwest coast of Europe, including the North Sea and the Norwegian Seas over the past few winter seasons.



Hindcast maximum one-dimensional wave spectra during Hurricanes Camille and Carla compared to measured maximum spectra generated by an extra-tropical cyclone in North Atlantic Ocean during December 1959.

oceanweather inc.

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FROM: VINCE CARDONE

TOTAL NUMBER OF PAGES 4 INCLUDING COVER PAGE

REPLY TO: VOICE - (203) 661-3091
FACSIMILE - (203) 661-6809

ADDITIONAL MESSAGE:

INTERPOLATION RECOMMENDATIONS ON ATTACHED SHEET FOLLOWED BY WORK SHEETS WHICH MAY NOT MAKE SENSE.

BASICALLY (WITH A FEW EXCEPTIONS) I USE WATER DEPTH WEIGHTED INTERPOLATION BETWEEN TWO GRID POINT WITHIN WHICH TRACK BEAR SAME RELATIONSHIP TO POINTS AS TO PLATFORM!

THERE ARE TWO EXCEPTIONS BASED ON MY JUDGEMENT.

Vince

ANDREW HINDCAST

Recommended grid points and interpolation weights

Y = interpolated hindcast variable at platform (e.g. HSmax)

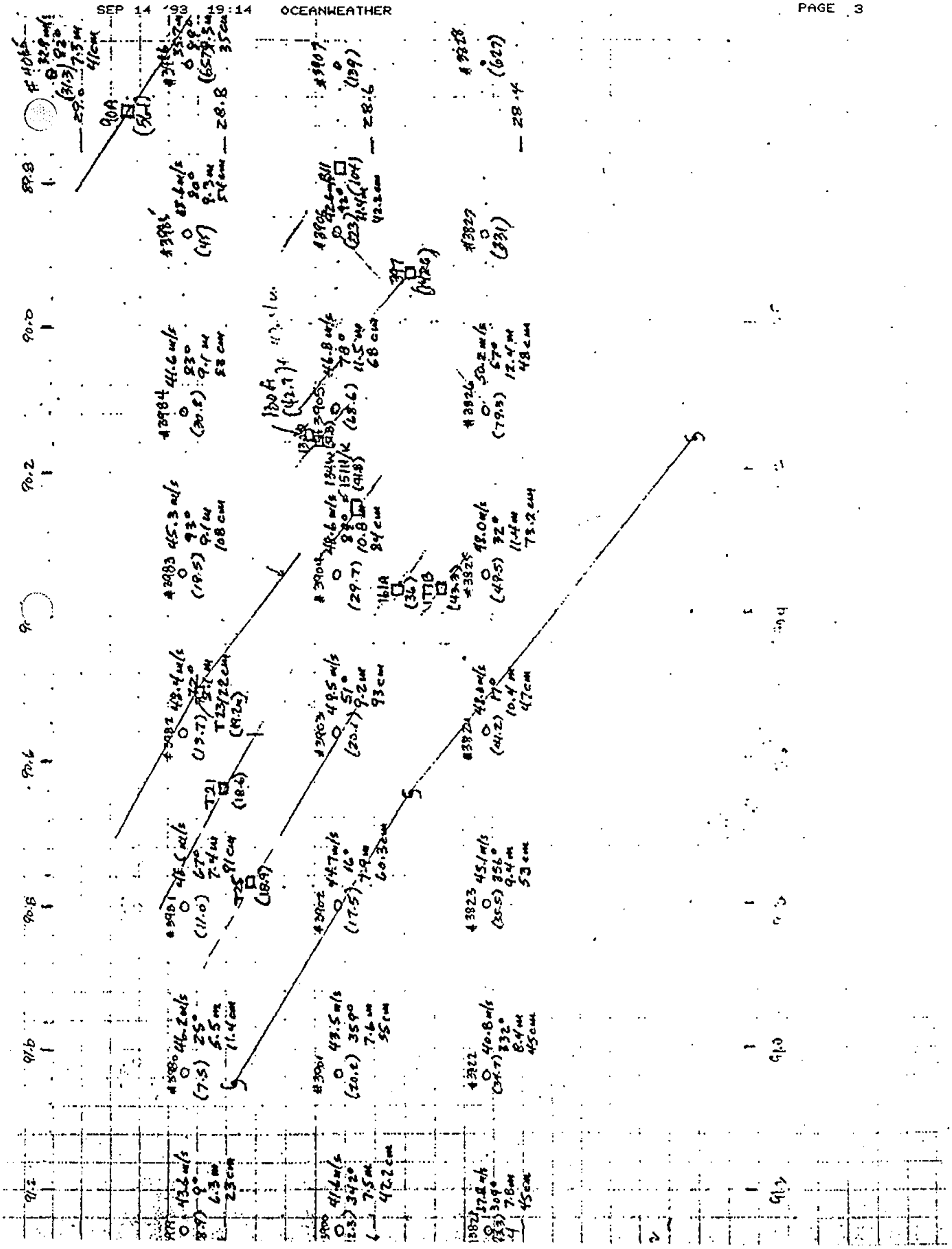
X_d = hindcast variable at grid point # d

X_s = hindcast " " " " # s

w_t = interpolation weight

$$Y = X_d - (X_d - X_s) * w_t$$

Platform	d	s	w_t – (water depth weight)
T25	3903		0
T21	3903	3982	.0625
T23	3904	3983	.920
177B	3825	3904	.280
161A	3825	3904	.650
134W, 151H/K	3905	3904	.670
130 Q	3905	3904	.410
130 A	3905	3904	.650
397	3826	3906	.625 (distance weight)
~~104~~ 311	3906	3985	.670
90A	3986	4065	.280



INTERPOLATION Recommendations:

GP#

125 #3903 — Bears exact relationship to track as platform; platform depth + surge = grid pt depth

72? #3982, 3903 — Platform depth 18.6m + 1m surge = 19.7m water depth, use depth weighted interpolation

$MOD_{plat} = MOD_{3903} - (MOD_{3903} - MOD_{3982}) * .0625$ — E.G. MAX HS = 9.1m

123 #3904, 3983 — Platform depth 19.2m + 1m surge = 20.3m w/d, use depth weighted interpolation

$MOD_{plat} = MOD_{3904} - (MOD_{3904} - MOD_{3983}) * .92$ — E.G MAX HS = 9.2 m

?B #3825, 3904 — Platform depth 43.3m + .7m surge = 44m, use depth weighted interpolation

$MOD_{plat} = MOD_{3825} - (MOD_{3825} - MOD_{3904}) * .28$ — E.G. MAX HS = 11.2 m

16A #3825, 3904 — 36 + .7m su = 36.7

$MOD_{plat} = MOD_{3825} - (MOD_{3825} - MOD_{3904}) * .65$ — E.G. MAX HS = 11.0 m

?W, S/H/K #3904, 3905 — 41.8m + .7m = 42.5m

$MOD_{plat} = MOD_{3905} - (MOD_{3905} - MOD_{3904}) * .67$ — E.G. MAX HS = 11.0 m

30Q #3904, #3905 — 51.8m + .7m ≈ 52.5m

$MOD_{plat} = MOD_{3905} - (MOD_{3905} - MOD_{3904}) * .41$ — E.G MAX HS =

3?? #3826, 3906 — 142.6m + .4m ≈ 143m — distance weight instead

$MOD_{plat} = MOD_{3826} - (MOD_{3826} - MOD_{3906}) * .625$

104 #3906, 3985 — 104 + .5 ≈ 104.5 m

$MOD_{plat} = MOD_{3906} - (MOD_{3906} - MOD_{3985}) * .67$

Appendix E
PF Program

The "bias factor, B," was alternately labeled as a "correction factor, C," earlier in the project and in the draft final report dated August 1993. The PF program code and its description (written early in the project) includes correction factor, C. Therefore, "C" should be read as "b" in the PF program description provided in this Appendix.

PROGRAM PF

GENERAL DESCRIPTION

Version 23. Friday 25th June, 1993

In its most basic form, the program computes the probability of failure of a platform subjected to given hindcast environmental conditions. As an extension it will compute the likelihood function given a failure. A further option allows the user to perform automatic parametric studies on the numerical integrations, to develop an optimized integration strategy. This optimization search can itself be automated, so that the program develops an accurate and efficient integration procedure.

The strength of the platform is assumed to follow a lognormal distribution whose median XMNR, and coefficient of variation, COVR are known.

The force on the platform from any individual wave can be estimated from the current U, and wave height HT, using the formula:

$$\text{Force} = C1\ (HT + C2\ U)^{C3}\ .\ E0$$

C1, C2 and C3 are three constants related to the geometry of the platform.

E0 is the error in this estimate of base shear. This error, E0, is assumed to follow a lognormal distribution with median 1 and standard deviation (of the logarithm) of SIGMAE0.

The loading on the platform comes from waves and currents estimated for NH time segments of a hindcast storm. For each of these segments there is an estimated significant wave heights HS(J), mean zero-crossing period TJ(J) and current strength U(J).

The significant wave height, HS, and current, U are assumed to be related to the hindcast values, HS(J), and U(J) by the expressions:

$$HS = HS(J)\ .\ E1$$

$$U = U(J)\ .\ E2$$

where these errors E1 and E2 follow lognormal distributions with medians XMNE1, XMNE2 resp., and coefficients of variation E1, E2, resp.

FORMULATION

The formulation of the computation follows that described by C. Allin Cornell in his faxes to PMB on Feb 8th and 10th 1993.

The probability of failure is described in detail in the Feb 8th fax, while the likelihood function was added in the Feb 10th fax.

From telephoned communications with Cornell, the functions $f_R(x)$, $f_{EHS}(E1)$ and $f_{EU}(E2)$ are lognormal functions, and the "Rayleigh-like" distribution $f_{H|HS}$ is Weibull. (Rayleigh is a special case of Weibull.)

The program involves four nested integrations, with respect to the following variables from outermost to innermost:

1) Base force, X, integrated from zero to infinity
2) Error in hindcast significant wave height, E1, integrated from zero to plus infinity
3) Error in hindcast current, E2, integrated from zero to plus infinity
4) Individual wave height, HT integrated from zero to a user-defined limit that reflects the maximum physical wave height that can exist at that site, as governed by the wave starting to break.

The integrations are all performed numerically, between user-prescribed lower and upper limits, using numerical integration at constant spacing between these limits. The integrations are performed in the following way:

The program starts to loop over the NINTX steps in the variable X, from the lower limit XLIML to the higher limit XLIMH. At each of these steps the value of X is determined and then to determine the integrand, the program descends to the next lower integration, which involves E1. Using the limits E1LIML and E1LIMH, and the NE1 steps specified, the program again starts looping over values of E1, but it again stops and descends to the next lower integration, continuing in this way until the innermost loop (on HT) is reached. The integration limits and number of integration points are as follows:

Base Shear	XLIML	XLIMH	NX
Error in hindcast Hs	E1LIML	E1LIMH	NE1
Error in hindcast U	E2LIML	E2LIMH	NE2
Individual wave height HT	HTLIML	HTLIMH	NHT

After reaching the innermost loop the program can then step through each value of HT, evaluating the integrand, until the innermost loop is completed. The program then ascends to the next higher loop and moves on to the next integration point in E2, again dropping down to the innermost loop to evaluate the integrand.

The integrations are theoretically performed over an infinite range, but the numerical integrations are performed between user-specified limits that must be chosen in such a way that ignoring the integrals outside of these limits does not affect the result significantly. These limits are specified by the user in multiples of the standard deviation on the lower and upper side of the mean value. The user also specifies the number of integration points, thereby specifying the step size. Since the computation does not take a trivial time for real situations, the limits and step size should be optimized, so that sufficient accuracy is obtained but in a reasonable execution time.

In all cases, should the user (or the program during automatic optimization) attempt to use a negative value of the independent variable in the integration, it will be reset to zero.

Further, should the user (or the program) attempt to integrate using wave heights greater than the input breaking wave height, this height will be set as the limit.

In resetting the lower or upper limits, the program adjusts the number of integration points to maintain the same spacing as the user originally specified.

Integration Details and the Parametric Variation Option

The following problems could arise in the choice of the integration limits and step size for any integration. (It will be observed that the three issues are inter-related.)

1) One or both of the integration limits are too close to the mean. In this case the integrand is not insignificant outside the region being integrated, and part of the integral outside the limits is lost, with resulting loss in accuracy. See fig.E1.

2) One or both of the integration limits are too far away from the mean. See fig.E2. In this case, although accuracy does not suffer, the execution time may be intolerably long. For a given accuracy, the region where the integrand is significant determines the step size. The integration steps outside this region are the same size and these computations are wasted since they do not contribute significantly to the integral.

3) The size of the steps is too coarse for the shape of the integrand. See fig.E3. In this case accuracy is lost in the integration. In an extreme case, the important part of the integration could be missed entirely. See fig.E4.

4) The size of the steps is too small for the shape of the integrand. Accuracy will be adequate, but execution times will suffer.

Because of the four-deep nesting of the integrations, execution times vary as the fourth power of the number of steps in each integration. If all levels have twice as many steps as necessary, the execution time will be 16 times longer than necessary, which could be unacceptably long. On the other hand, we do not want to risk coarse integration steps or too narrow an integration region, or the accuracy will suffer, perhaps greatly.

The following procedure is recommended to ensure a balance between accuracy and length of executions.

The program is able to operate in three modes, two of which interest us at this point. The first is the computation of the probability of failure. The second is an automatic parametric study on the integration procedure, that gives the user valuable insights into the integration of each loop. (This can be followed by automatic modification of the integration procedure, and re-analysis. In this way the program can be left by itself to develop an optimum integration procedure. This will be discussed below.) If the user requests this parameter study, the program will re-execute the problem specified by the input file 24 times, varying, in each execution, one of the parameters that determine the integration limits or the integration step size.

In the first 12 re-analyses, a constant multiplier on these integrations is used, and in the second 12, another is used.

Specifically, for all of the integrations, the following 12 variations are performed.

1) Change the lower limit, (defined by the number of standard deviations below the mean). The integration step size is kept as close as possible to the base case execution.

2) Change the upper limit (defined in the same way above the mean). The step size is again kept close to the base case.

3) Change the number of integration points, keeping the base case limits.

Since there are four integration loops, this represents a total of 12 re-analyses.

Two sets of 12 runs with changes from the base case are made using two factors specified by the user. These would normally be a decrease and then and increase, perhaps of 0.5 and 2.0. At the end of the execution a table is printed that shows what percentage change in the probability of failure was caused by each parametric variation.

A procedure to develop an efficient accurate integration strategy may include the following.

Create a base case strategy, by adopting the low and high limits about 2 - 4 standard deviations below and above the mean, and choosing about 7 - 15 integration points for all except the lowest (wave height) loop, which should be about 101 - 201.

The probability of failure is extremely sensitive to the accuracy of the wave height integration, because the results of this integration are raised to an exponent in the loop over hindcast seastates that is equal to the sum of the number of individual waves in each of the time segments hindcast. This exponent will typically be of the order of 350 for each hour, of 1300 if 4 hours are used. For this reason a large number of integration points are required for the wave height loop, and this number depends very much on the number of hours of the storm.

For this reason high accuracy must be maintained in the computation of the Gauss (normal) distribution in the lowest integration loop. The CDF is not found by numerical integration, but by a polynomial approximation. Three algorithms are available, one with a cubic polynomial, another with a quintic and the last with a sextic polynomial. The first two use the PDF as part of the computation. (See Handbook of Math. Functions, Staff of Research and Education Association.) The last version depends solely on the 6th order polynomial. It is recommended that the 5th order polynomial be used since this is supposed to be the most accurate (by specifying a 5 in the input data at the appropriate place.) This will result in some loss of execution speed relative to the 3rd order algorithm. The 6th order polynomial was slightly faster than the 5th in some tests, but it is supposed to be slightly less accurate.

Run the parameter study using factor of 0.5 and 2 for the two sets of 12 re-analyses. It will be explained below how to set these factors.

Examine the changes in the probability of failure and see whether sufficient integration has been adopted in the base case that increases have not changed the result appreciably. If only small changes result, the computation is presumably accurate enough. Then check that integration parameters been chosen such that decreases in integration parameters do not alter the results much. If this is the case the integrations can perhaps be reduced.

Since the process is nonlinear, the optimal integration strategy for one integration will be affected by increasing the accuracy in the integration of any of the other integrations, whether from more inner or more outer integrations.

Eventually, an optimal set of limits and integration spacing will be arrived at. When this is the case:

1) An increase of 2x (or more) in all parameters will <u>not</u> affect the probability of failure, indicating the desired accuracy has been obtained. This is a crucial criterion.

2) A decrease of 0.5 <u>will</u> have a significant effect on the probability of failure, indicating that you cannot reduce the execution times much by reducing the integration procedure. This is only important in as much as it keeps execution times low.

Automatic Optimization Feature

The program has an automatic optimization feature that allows the program to perform essentially the steps discussed above. When this option is used, the program will perform the 24 parameter studies, the first 12 of which should be with reduced parameters and the second 12 with increased. Then, on the basis of the changes in the probabilities of failure, it will adjust the integration parameters for future analyses. It then re-runs a base case with the new parameters and then does more parameter studies only on the parameters that were adjusted. In this way the program will converge on an accurate, but efficient integration strategy.

An option exists to get the program to only change a user-specified number of parameters at a time, at the most NOPTD decreases in optimization parameters and at the most NOPTI increases. This would allow you to only increase the optimization by specifying NOPTD as zero.

The percentage tolerances that the program will accept without modifying the integration parameters, can be set by the user. These are PDMIN and PIMIN for decreasing and increasing integration parameters. Further, the factors that will be used for the parameter studies are also user-defined as FDMIN and FIMIN for the decreasing and increasing parameters. These are augmented by user-defined variables PDMAX, PIMAX, FDMAX and FIMAX. These are all now described.

In the automatic mode, the program first performs a base case analysis with the parameters input by the user. This is noted in the output as the parameter or base case 0.

It then decreases all 12 of integration parameters sequentially, by the factor FDMIN, moving along the rows and then down the columns of the following matrix, as shown

	Low limit (Sigmas)	High Limit (Sigmas)	Number of Points
Base Shear	1	2	3
Error on Hs	4	5	6
Error on U	7	8	9
Wave Hts	10	11	12

It then increases all 12 of integration parameters sequentially, by the factor FIMIN, moving along the rows and then down the columns of the following matrix, the count being as shown

	Low limit (Sigmas)	High Limit (Sigmas)	Number of Points
Base Shear	13	14	15
Error on Hs	16	17	18
Error on U	19	20	21
Wave Hts	22	23	24

The percentage change from the base case is then recorded and printed for each of the 24 parameter runs. If this change is greater than PIMIN for any of the 13 - 24 parameter runs, that parameter is adequately large for accuracy, and the corresponding run 1 - 12 is checked. If that change is less than PDMIN (say it is PD) then the parameter is large enough for accuracy but is unnecessarily large for efficiency. So this integration parameter can be reduced by multiplying it by a factor less than FDMIN. It is reduced by a (nonlinear) interpolation procedure as shown below.

PDMIN (say 1%)	PD (say .03)	PDMAX (say .001%)
FDMIN (say .5)	FD (.35)	FDMAX (say .2)

Thus, if the parameter study was made by reducing integration parameters by a factor of 0.5, and the tolerance was set at 1%, and we found that the % change for a particular parameter is .03, it is clear that we can reduce the input parameter by more than the 0.5 factor, in fact by 0.35 according to this algorithm. If the % change was less, the original integration was correspondingly more conservative and an even smaller factor would be used.

If, when the runs 13 - 24 were made, it was found for a particular parameter that the change was smaller than our tolerance, but on reducing this parameter (corresponding run in the 1 - 12 set) the change was large, this shows that the integration was adequate for convergence, but cannot be reduced without affecting accuracy too much.

The last possibility is if the runs 13 -24 for a particular parameter show a % change greater than PIMIN (say 50%). In this case the integration is not enough and an increase must be made.

PIMIN (say 1%)	PD (say 50)	PIMAX (say 1000%)
FIMIN (say 2.0)	FD (3.7)	FIMAX (say 5.0)

Interpolation from the above values would give a multiplier of 3.7.

The program assesses each integration parameter as just described, assigning it a multiplication factor FD which may be less than, equal to or greater than FDMIN and FIMIN as explained assigning the new parameter as a base case. If the user selects NOPTD or NOPTI as less than 12 it will find the NOPTD decreases and the NOPTI increases that make the most difference, otherwise is find all parameters that need changing. It then redoes the parameter study from this base case with the FDMIN and FIMIN multiplication factors. However, in this second parameter study (and in all following iterations) it only does the parameter runs for those parameters that were modified after the first base case. This assumes (incorrectly sometimes) that parameters that were ok in the early stages of optimization will remain adequate after those that needed changing are adjusted.

This process is then repeated checking parameters that were not adequate, in a sequence of new base cases, followed by parameter runs on some parameters, until convergence is obtained.

After this, the program either stops or goes on to the next value of c for the likelihood function calculations. In the latter case, it uses the current base case that has just been tested and found to be adequate for the base case for the next c, to avoid starting the whole process again from the (presumably inadequate) user's first estimates.

Recommended Automatic Optimization Procedure

A suggested plan of attack is the following:

Start of with a reasonable estimate of integration parameters, for instance:

	Low limit (Sigmas)	High Limit (Sigmas)	Number of Points
Base Shear	3	3	15
Error on Hs	3	3	15
Error on U	3	3	15
Wave Hts	2	3	91

(Note that if the low limit is too many sigmas, so that the actual value becomes less than zero, the program will correct for this and reset it to zero, with the appropriate number of sigmas. Similarly if the high wave height is greater than the breaking wave, the program will reset it.)

Define the tolerances PDMIN and PDMAX equal and in the neighborhood of 1% - 5% depending on the demands of the use of the information derived. Set the factors for multiplying the base cases in the parameter studies, FDMIN and FIMIN. Set the other factors PDMAX (< PDMIN), FDMAX (<FDMIN), PIMAX (>PIMIN), FIMAX (>FIMIN). (Default values can be used.)

Decide on a limited number of values of C to guide the later detailed likelihood function evaluation. Perhaps 3 is adequate. Start from low values and end on high values, for the reasons given later.

Start the program looping over these values, allowing it 12 decreased and 12 increased parameter changes. (Sometimes it has been found that the decreases move too fast relative to the increases and integrations of 3 or 1 are obtained, which are clearly too few. This can be slowed down by increasing FDMAX to be closer to FDMIN. Another way is to set NOPTD to zero, preventing any reduction in parameters.

The program will then perform the base case and the 24 parameter runs. It will then analyze the changes from the base case in the parameter runs and assign a set of multipliers to create the next base case. If the original user values were adequate, these will all be 1.0 and the program will move on to the next value of C. If any parameters were judged too low or too high, they the corresponding multipliers will be printed and the base case will be modified accordingly.

The program will then perform the new base case and new parameter variations for those that were changed. Based on these new changes relative to the new base case further iterations of this base case - parameter study will be performed. Eventually all the multipliers will be 1.0 indicating that the program has found convergence for the last base case. It then moves on to the next C value with the parameters found in the last base case.

It then starts the process entirely fresh, running a new base case and 24 variations, and seeking the optimal integrations for this new value of C. When this is found it moves onto the next C.

The program only checked parametric variations on those parameters that showed inadequacy in the first base case run. It is possible that after convergence there could be other parameters that were not originally a problem that are now. To be absolutely sure of this, the final strategy should be then rerun as a base case, with a full 24 run parameter study. If it passes this test, it is a good solution. If it fails, the user may decide whether to accept the errors shown. A quick way to get data for this final check run is to take the file called *scratch* which is the input file corresponding to the last base case (of the last C value.)

The final part of the analysis is to perform the full likelihood function computations at appropriately fine C spacing. By looking over the output file from the optimization run, the most critical values of all 12 parameters can be found. These should be used.

Likelihood Function

In addition to computing the probability of failure, the program is also able to determine the likelihood function, given a failure. This function can be determined over a range of the independent variable c chosen by the user. The number of points at which the function is evaluated is also a user-specified parameter.

In calculating the likelihood function, it can easily happen that the integration strategy that was set up for the computation of probability of failure is not entirely suitable for the expanded range of calculations, and the same problems that were examined in previous section can occur. For this reason, the program can perform a parameter study on the integration strategy using the computations for the likelihood function.

It is suggested that the user perform parameter studies on the likelihood function computation using two or three values of c, the lowest, the highest and maybe the middle value. If the changes in the values of the likelihood function for a given c from the parameter study change by more than a percent or so, choose a new integration strategy that incorporates the possible problems of the high and low c values, and repeat the run. An acceptable result will be when the parameter study with factors of 0.5 and 2.0 does not change the results for a particular value of c by more than a few percent. This can also be automated as described above for the single probability of failure calculation.

The final runs for the finer spacing of the variable c can then be set with integration parameters that envelope the values found to be necessary at the two or three values of c chosen for the optimization runs. It has mostly been found that the most demanding integrations occur with the highest values of C, so if C values increased in that run, the file *scratch* can be used directly for the final check run. This simplifies batch processing.

It should be observed that each data set has its own characteristics and no final guidelines will necessarily work smoothly on all data sets.

Individual Wave Height Distribution

The wave height distribution is a Weibull function. In its most general form, this is a two-parameter function defined by two parameters a,b.

CDF: $P(H) = 1 - \exp(-aH^b)$

PDF: $p(H) = abH^{b-1}\exp(-aH^b)$

The exponent b is set by the user. The parameter a is defined from the significant height H_s by the user-prescribed parameter a_1, from the relationship:

$$a = a_1/H_s^b$$

Thus $P(H) = 1 - \exp\{-a1(H/H_s)^b\}$

If b = 2, the parameter is the Rayleigh distribution.

In the definition of the Weibull distribution, for a given seastate we can define a and b independently. This fully defines the probability density function, pdf and cumulative distribution, cdf. From these we can determine the mean of the 1/3 largest waves, $H_{1/3}$. This will not in general be equal to the input value of H_s from which a was found. This is because H_s is defined by:

$$H_s = 4\ \sigma_\eta$$

not by the mean of the largest 1/3 waves $H_{1/3}$, and cannot be found from the cdf or pdf without further assumptions about the free surface elevation distribution.

For a given value of b there is only one value of a for which H_s will be equal to $H_{1/3}$. For instance, for the Rayleigh function this only occurs when

$$a = 2.005.../H_s^2$$

Special distributions of interest are:

1) Classical Rayleigh. b = 2 $a = 2.005.../H_s^2$
$H_s = H_{1/3}$ is true here.

2) Rayleigh. b = 2 $a = 2.0/H_s^2$
This is close to the classical Rayleigh, but in this case, H_s is defined from $H_s = 4\ \sigma_\eta$

3) Longuet Higgins. b = 2, $a=2.337/H_s^2$
See Journal Geophysical Research, 1980, pp 1519-1523
This was the result of trying to fit measured data.

4) Forristal. $b = 2.126$ $a = 2.263/H_s^{2.126}$
See Journal Geophysical Research, 1978, pp 2353-2358
The result of fitting measured data also.

Outputs from Program

The outputs from the program consist of:

1) Output.

This shows the original input, and the probability of failure from this, the base case. Then it shows the modifications to the integration parameters for the parameter study runs, and the resulting probabilities of failure. During an optimization run, the results of the parameter study are collected showing the probability of failure for each parameter change run, and the percent change from the base case. The changes in these parameters are then displayed, and then the results of the next parameter runs starting off with the new base case parameters.

For a likelihood function study, the above results are repeated for each value of c. Finally at the end, there is a table of the probability of failure vs c, the values being taken from the base case runs for each c value.

2) File named *scratch*

This is an input file similar to the one input by the user, but it is rewritten for every new parameter run case during optimization or likelihood function evaluation. Since the output file does not always display information that is current, due to buffering of output by the machine, this file can be read to see what the program is up to.

3) Files *loop.bs, loop.e1, loop.e2, loops.h*

These files show the cumulative distribution function CDF, the probability density function PDF, and the product CDF*PDF in each level of the integrations. *bs* is the upper loop over base shear, *e1* is over the error in hindcast significant wave height, *e2* is over the error in hindcast current, and *h* is the individual wave height.

Since the program loops over these integrations hundreds of times with different values of the CDF and PDF in each loop, one particular loop has been selected for each file. This occurs when the independent variables in the loops above the level being displayed are at the value closest to the mean value. Specifically, therefore the following applies:

loop.bs is unique and is printed for each value of base shear.

loop.e1 is printed for the loops of *bs* when it is at its mean value.

loop.e2 occurs for loops of ***bs*** and *e1* both at their mean value.

loops.h occurs when loops for ***bs***, *e1* and *e2* are at their mean values. There may however be more than one seastate in the loop over seastates, so, rather than choosing one or the other, the data is printed for all. This file therefore contains as many sets of data as there are seastates.

The data consists of a table for which each row has the independent variable, the CDF, the PDF and the product CDF*PDF in 19 column fields. This table is preceded by a header that can be used with PSgraph program. The first two lines of the header are to be retained, and the next two headers for PSgraph files are taken from either of the three pairs following, for either CDF, PDF or CDF*PDF. With a minor amount of editing all three can be displayed on one graph. See PSgraph input file requirements.

It should be emphasized that to data for any loop correct, all the lower loops need to have been integrated accurately. Thus to get the base shear table, the full integration must have been done. At the other extreme, to get the wave height table, the integrations for the upper loops can all be set to 1 integration point.

4) Screen

The program displays essentially the same information on the screen as on the output file. It also gives an estimate of the execution time of the base case and one round of 24 parametric runs. It shows as the program is running the percentage completion of the current evaluation of the probability of failure.

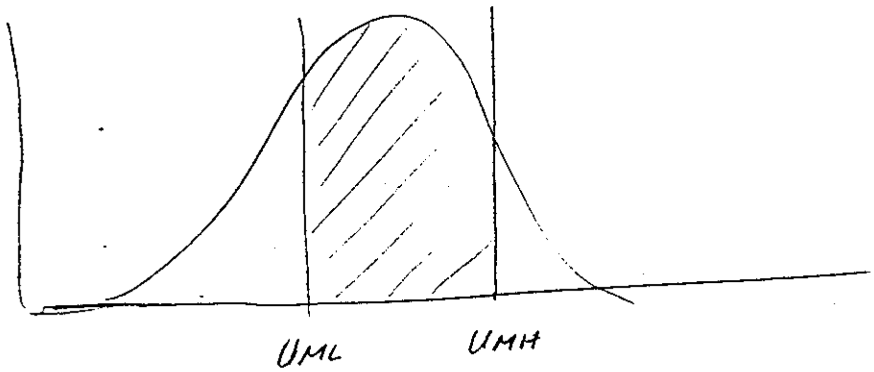


FIGURE-E1

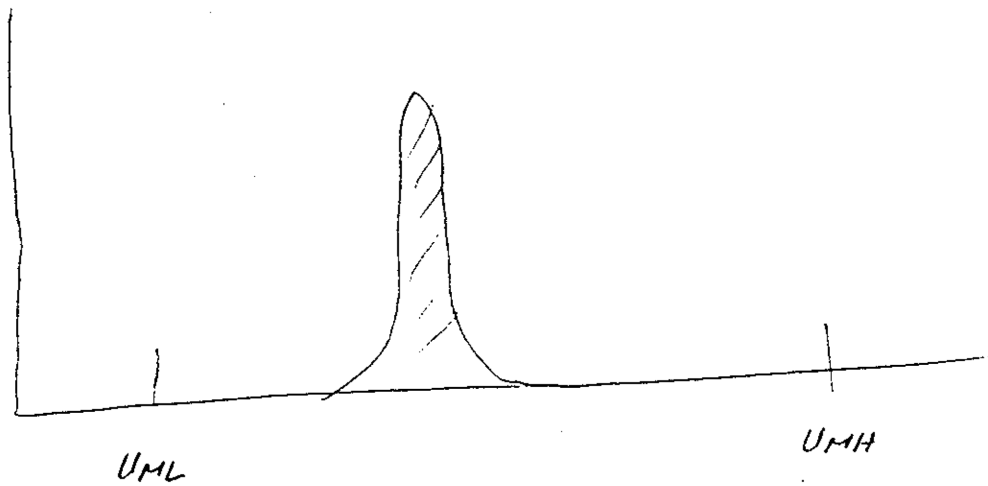


FIGURE-E2

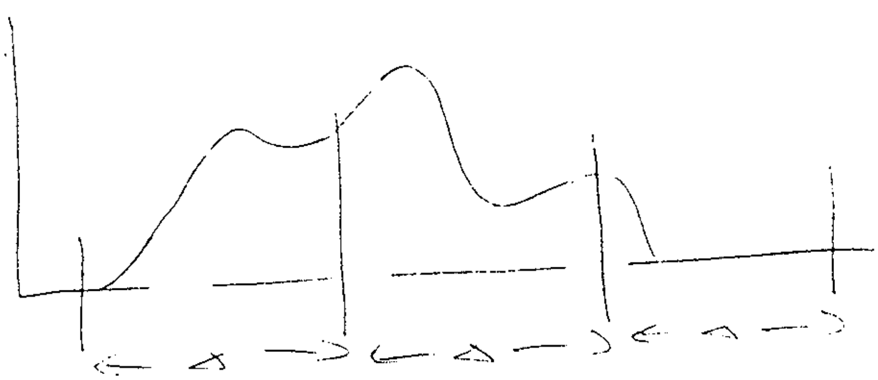


FIGURE-E3

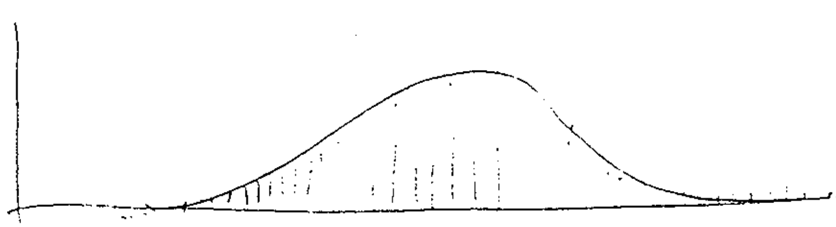


FIGURE-E4

Appendix F
C1C2C3 Program

PROGRAM C1C2C3

GENERAL DESCRIPTION

The program determines best fit values for the constants C1, C2 and C3 in the equation:

$$BS = C1(H + C2.U)^{C3}$$

representing the base shear, BS, on a platform from a wave H, and current U. The data being matched may be one of two options:

1) Three or more sets of data BS,H,U from measurements or wave loading simulations.

2) Three or more sets of data BS, H from measurements or wave loading simulations, plus a relationship between H and U provided in a table. If this option is used the currents corresponding to each of the wave heights H is found by interpolation from the H,U table and then the computation proceeds exactly as for the other option.

The constants C1, C2 and C3 are related to the geometry of the platform. C3 is dimensionless, but C1 and C2 depend on the units used for H, U and BS.

FORMULATION

The formulation of the computer program is based on determining the values of C1, C2 and C3 that result in a least squares best fit to the data supplied.

The data consists of NPTS data points the i^{th} of which consists of a wave height H_i, a current U_i, and a base shear BS_i. For a given set of values C1,C2,C3 the base shear can be evaluated from the above equation, and the difference between this and BS_i is the error associated with this data point and this set C1,C2,C3. This can be written:

$$E_i = BS_i - C1(H_i + C2.U_i)^{C3}$$

The measure of error for all the data points is found by squaring and adding:

$E = \sum E_i^2$ the summation being made over the NPTS datpoints.

For any set of values C1,C2,C3 there exists a value of this error, E. The problem is therefore to find the values of C1,C2,C3 that minimize this error.

The procedure used is the downhill simplex method in the three dimensions C1,C2,C3. The method will be explained in two dimensions, since this is more easily visualized.

The error E is a surface that is a function of the two variables C1 and C2. We wish to find the values of C1 and C2 that minimize the function E. We start at some point (C1,C2) near the solution and follow the downhill direction until the difference in E from successive positions is very small, which means that we have achieved a minimum. We must start at a value near enough to the solution that the procedure does not follow the wrong valleys to a local minimum that is not as small as the global minimum we seek.

The procedure starts with a triangle consisting of three points in the (C1,C2) plane. This triangle is then moved downhill in series of steps, until it reaches the minimum. At each of the triangle vertices, the function E is determined, and the vertex with the largest value is identified. It is then assumed that the downhill direction passes through this point and the middle of the opposite side. The two lower corners of the triangle remain as corners of the next triangle, but the high corner is then moved to a point outside the triangle on the downhill line previously identified. Then the process is repeated with the new triangle. The triangle thus slithers downhill in a series of steps to a minimum value of the function E.

Actually, this problem is three-dimensional, so a tetrahedron is used instead of a triangle, and the downhill motion is from the vertex with the maximum function value through the center of the opposit face.

As mentioned above, the minimum that is identified may not be a global minimum if the initial vertices of the tetrahedron are in the "wrong valley" from the global minimum. The only safe way to ensure that we have the correct global minimum is to rerun the solution with a series of different starting points, and check that either

(1) the analysis converges to the same values of C1,C2 and C3

(2) the analysis converges to a different solution with a larger root mean square error

The iterating sometimes may not stop even though a good solution has been found and the solution oscillates backwards and forwards in the minimum without ever achieving the desired small change in the function between steps. When this occurs, the iterations can be stopped and the solution checked. If the the accuracy of the solution is high enough, as it generally seems to be, there is no problem.

USE OF THE PROGRAM

The user can supply three or more sets of (BS,H,U) triplets that represent the manner in which the base shear, BS, varies with wave height, H, and current, U. Alternatively, the input can consist of (BS,H) pairs, followed by a separate table of (H,U) pairs, from which the full data set of (BS,H,U) values can be filled out by interpolation.

The user also provides four sets of (C1,C2,C3) that become the starting values for the interation. These values may be explicitly input by the user, or defaults may be used. The values must be in the neighborhood of the correct ones, or convergence to the wrong solution may result.

After finding the values of C1,C2,C3, it is recommended that the user check that the solution is reasonable. This can be done in two ways. The first is by examining the output to see how big the differences are between the original data points, and the base shears computed from C1,C2,C3. Alternatively, the variation of base shear with wave height could be plotted for a low and high value of the current, and the input data can be added to this plot. To assist the user, the program generates a file callled graph that can be plotted using PSgraph on the SUN, or it can be imported to a spreadsheet. In either case, if the solution looks good, the results can be accepted, otherwise the problem should be rerun with a new starting point seeking a better solution.

Appendix G
Load and Resistance Recipe

Appendix G

LOAD AND RESISTANCE RECIPE FOR CALIBRATION

This Appendix includes a summary of the criteria used for evaluation of loads on and resistance of candidate platforms selected for the Andrew JIP. The load and resistance recipes presented in the following sub-sections were used for the calibration of the pushover analyses.

LOADS PER API RP2A 20th Edition:

Loads on the platform have been evaluated by the new guidelines presented in the 20th Edition of API-RP-2A. These loads may differ from those obtained by previous editions.

1.1 Gravity

a. Structural Framing

- Material Weight: Steel: 0.4905 kips/ft3
  Grout: 0.1370 kips/ft3 (dry weight)
- Flooded Members: All main legs, skirt legs, casings and conductors will be considered flooded, unless otherwise identified for a candidate platform.
- Buoyant Members: All jacket members except legs, skirt legs, casings, conductors considered buoyant, unless noted otherwise.
- Marine Growth 0.075 kips/ ft3 (dry weight of hard marine growth)

b. Deck Loads

- Specific equipment weight: dead loads
- Distributed loading: 25 % of live load during hurricane

Equipment loads which are applicable during hurricane loading were considered. The equipment load distribution on deck legs were estimated and used in analysis. No detailed distribution of equipment loads were considered.

1.2 Wind

a. Wind plus hurricane wave condition only were considered in the evaluation of the total wind loads on the deck equipment and structure.

b. Wind speed

- Hindcast: per Oceanweather data.

- 30 min to 1 min

- Gust factors: per Oceanweather

c. Wind loads: per API formulation, Eqn. 2.3.2.8

d. Shape coefficients: per API Section 2.3.2e

e. Shielding coefficient: per API Section 2.3.2.f

1.3 Waves

a. Wave height

- Hindcast: per Oceanweather data

b. Wave period

- Hindcast: per Oceanweather data

c. Storm surge plus tide

- Hindcast: per Oceanweather data

d. Wave directions

- Hindcast: per Oceanweather data

e. Wave loads per Morrison equation w/ API corrections

- Stream function profile and kinematics:
  Stream function wave theory (Dean and Perlin, 1986) of 3rd order was used for intermediate and deep water locations. In case of shallower water depths, higher order of Stream Function theory was used in accordance with Fig. 2.3.1-3 of API 20th edition.

- Wave kinematics correction factor:
  Wave kinematics correction factor of 0.88 was used to account for wave directional spreading and irregular sea effects.

- Combined wave / current kinematics:
  wave kinematics adjusted for directional spreading and irregular seas were combined vectorily with the stretched current profile.

- Marine Growth:
  In cases where actual marine growth profiles are not available from inspections, the general profile given in Section 2.3.4.d of API 20th edition for the Gulf of Mexico was used.

Elevation	Marine growth (inch)
Above MHHW	None
MHHW to (-)150'	1.5
Below (-)150'	none

- Drag (Cd) and Interia (Cm) Coefficients:
  For unshielded circular cylinders with K-C number (=UT/D) more than 30 the following coefficients were used:

Type	Cd	Cm
smooth	0.65	1.6
rough	1.05	1.2

- Conductor Shielding Factor:
  Wave force reduction factor in accordance with Figure. 2.3.1.4 of API 20th Edition were applied to the drag and inertia coefficients for the closely spaced conductor arrays.

- Appurtenances:
  Jacket appurtenances include boat landings, fenders or bumpers, walkways, stairways, grout lines, and anodes. The hydrodynamic loads on only major appurtenances in the wave zone such as boat landings, fenders and bumpers was determined.

1.4 Currents (with waves)

a. Surface velocity
Hindcast: per Oceanweather data
Variation with depth: per API 20th Edition

The "free field" current profile was used.

b. Current direction
Hindcast: per Oceanweather data

c. Structure effect on current velocity
Current blockage factor:
The blockage factor for current direction was obtained from Section 2.3.1.b-4 of API 20th Edition. The effective local current profile was determined by multiplying the free field current profile with current blockage factor.

d. Current profile stretching:
The current profile was stretched to the local wave surface by vertical stretching in case of slab current or by linear stretching for other current profiles, as per Section 2.3.1.b-5 of API 20th Edition.

1.5 Wave in Deck

a. API TG 92-5 preliminary procedure (Table 3-1).

RESISTANCE PER API RP 2A 20th EDITION

2.1 Deck

a. Material classification per operator

- In general, adjusted yield strength of primary and secondary members was used (42 ksi in the case of 36 ksi steel).

- The mill certificate or field test data was used, when available.

b. Primary members only: The primary members (deck legs, deck girders and deck trusses) were modeled in detail and the secondary members were represented by equivalent sections to simulate load paths. Secondary members such as deck beams and stringers, plating or grating, and cantilever support framing were represented by X-braces to transfer load between deck legs and deck girders.

c. Non-linear elements: Deck legs were modeled as non-linear beam columns.

d. Linear elements: Deck braces, deck girders, and deck trusses were modeled as linear beam elements, unless their failure is apparent. Equivalent braces for secondary members were modeled as linear beam elements.

2.2 Jacket

a. Material classification per operator

- Adjusted yield strength (42 ksi)

- Mill certificates or field test if available

b. Legs/Piles

- Nonlinear beam/columns

- Effective length factor, K as per API-RP-2A. K = 1.0

- Leg/pile annulus grouted:
  Composite leg/ pile section properties were used. Equivalent section properties (A, I) were evaluated to account for steel sections of leg and pile, as well as the marginal effect of leg can sections (if provided). The material properties were based on the main leg sections and piles. In case leg and pile actual yield strengths differ, the lower values was used or an equivalent value was determined.

- Leg / pile annulus ungrouted:
  Explicit leg and pile with shims were used. Equivalent section properties (A, I) for the legs were evaluated to account for the marginal increase in leg properties due to leg can sections (if deemed appropriate).

c. Braces

- Struts (pure axial with compression buckling):
  The braces in vertical frames of jacket and the braces in horizontal frames, which span between legs and carry primarily axial loads and are likely to fail by buckling were modeled as Marshall Struts. The material properties of the struts will be evaluated considering effect of marine growth and lateral wave loads..

- Buckling Beams (axial plus bending):
  The braces in horizontal frames which are likely to yield because of moment hinge formation will be modeled as nonlinear beam/columns.

- Allowable Capacity:
  The allowable capacity of braces is based on the API RP 2A LRFD equation D.2.2-2.

- Effective length factor, K:
  see page 3-3

d. Joints: In general, the ultimate strength of leg- brace joints will be computed and compared with the ultimate strengths of braces (strut, beam column).

- Model within brace:
  If the joint capacity is lower than brace capacity, its effect will be accounted for in the brace capacity using a nonlinear elastic-plastic truss element.

  Ungrouted joint capacity: Per API RP 2A, without factors of safety

- Grouted joint capacity:

e. Secondary Members

- Include strength where applicable:
  The primary members of jacket launch trusses will be modeled. The conductor guide framing will be modeled by equivalent members to represent load path for load transfer between primary jacket frame members.

- The strengths of other secondary members and appurtenances will be ignored.

2.3 Foundation

a. Soil properties: as per operator or participants

b. Explicit non-linear pile-soil interaction: Non-linear pile-soil interaction curves per API RP 2A.

c. Vertical Capacity: The effect of cyclic storm loading on the soil was considered by using fully degraded soil properties.

d. Lateral Capacity: The full static capacity (cyclic degradation neglected) per recent laboratory work by EXXON.

e. Mudmat effect: Mudmat effect was not considered.

2.4 Conductors

a. Linear beams: Conductors were modeled with linear beam elements.

b. Lateral supports at guides: Conductors were modeled to move freely in the vertical direction and to transfer lateral wave loads in orthogonal directions to the jacket structure.

c. Explicit foundation: The conductors below seabed were be modeled as nonlinear beams with non-linear soil springs supports.