Andrew Foundation Study

Further Evaluation of Offshore Structures Performance in Hurricane Andrew

Development of Bias Factors for Pile Foundation Capacity

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San Francisco, CA

May 1995

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Executive Summary

Hurricane Andrew (hereafter called Andrew) was a very intense storm that passed through the Gulf of Mexico on August 24, 25 and 26, 1992. While most of these Gulf of Mexico platforms were not adversely impacted by Andrew, several were significantly damaged 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. The Andrew JIP was completed in October 1993.

The Andrew JIP predicted failures of foundation elements in a majority of platforms, which were not observed during post-Andrew inspections. The Andrew JIP established an overall (system) correction factor, with a mean of 1.2. This correction factor was not specific to the biases associated with the foundation capacity estimates per the "recipe" followed in that project. Rather it included failures of all types within jacket and its foundation. Therefore, to further investigate the bias in the foundation capacity estimates for the steel jacket platforms, API and MMS commissioned this study specifically to evaluate the effects of Andrew on offshore platform foundations. This is the final report, which documents the approach and results of the project.

This project had two primary objectives:

1. **Calibration.** Perform a calibration of procedures for foundation capacity analysis (lateral and axial) for assessing existing platforms. A rigorous method for calibration developed in the Andrew Phase I is to be used. The process includes reconciling analytically predicted platform damage and failure with observed field performance during Andrew.

2. **Factors Influencing Bias Factor.** The various parameters which would influence the bias factor are to be identified.

Three steel jacket platforms and three caissons subjected to hurricane Andrew were selected for evaluation in this project. The selection process considered previous Andrew JIP results, hindcast information and available structural and geotechnical information. The caissons provided a means of isolating the lateral soil behavior and some observed foundation failures. Figure ES-1 shows the locations of platforms and caissons relative to the path of Andrew.

GEOTECHNICAL ASPECTS

This portion of the project involved identifying various parameters which would affect the platform capacity estimates. In particular, factors influencing pile foundation axial and lateral capacity estimates were identified. In the case of lateral capacity, the API static p-y curves were considered appropriate to be included in the capacity analysis "recipe" by the API Steering Committee and the project team.

As the project progressed, it was considered that due to large uncertainty in estimates of the contributions of various factors (such as loading rate and cyclic effects) to the axial capacity, there will be no attempt to interpret the effect of individual factors on the pile capacity analysis recipe. Therefore in this project the combined effect of these parameters will be reflected in the bias factors.

CAPACITY ANALYSIS

The capacity analysis results obtained in the Andrew JIP indicated a number of possible failure modes in the jacket superstructure and foundation, when subjected to Andrew loads. In order to identify the differences in the biases associated with the pile lateral and axial capacities from those with the jacket superstructure, three separate analysis cases were carried out for each jacket platform:

Base case analysis:	Nonlinear jacket and foundation model — To determine critical failure mode for best estimate deterministic model.
Case 2 analysis:	Linear jacket and nonlinear foundation model — To determine critical foundation failure mode for best estimate deterministic model. The analysis may be repeated to suppress one foundation failure mode type if both lateral and axial modes are predicted.
Case 3 analysis:	Case 2 model with one foundation failure mode type (pile pullout/plunging or pile yielding/plastification) suppressed — To determine capacity in the alternate foundation failure mode.

The base case capacity analysis provided the results without eliminating the foundation effect. The Case 2 and Case 3 analyses provided estimates which define the ultimate foundation capacity for both lateral and axial pile failure modes individually. These three cases enabled the determination of the relatively uncoupled estimates of foundation capacities for use in calibration.

CALIBRATION

The bias (or correction) factors that are the focus of this study represent the modeling errors associated with the ultimate lateral and axial capacity estimates of the jacket foundation system. These correction factors reflect the bias of the overall safety factor (resistance divided by loading effects) for platforms during extreme hurricane loadings. A bias factor greater than 1.0 indicates that the current platform ultimate capacity analysis procedures provide conservative results in the sense that it suggests more failures than will actually occur during storms.

Two of the platforms studied were damaged during hurricane Andrew, specifically damage in the jacket frames. Unfortunately, limited information was available on the foundation behavior of the jacket platforms during Andrew, suggesting that they did not fail or deform in a noticeable way. Therefore, the capacity related to the jacket superstructure (with known behavior during Andrew) shall be considered separately from the foundation behavior. The geotechnical investigations indicated that the biases associated with the foundation system are significantly different between the lateral and axial failure modes. Thus, these two modes shall also be considered separately. In summary, it is expected that there will be different biases in the following three failure modes and corresponding capacity estimates:

- Bias in the jacket superstructure, B_{ss}
- Bias in foundation lateral capacity, B_{fl}
- Bias in foundation axial capacity, B_{fa}

In this study, the foundation bias factors (B_{fl} and B_{fa}) were determined utilizing the capacity estimates for the Case 2 and Case 3 capacity analyses.

The analysis results indicate that foundation failures (lateral and axial) can occur at relatively small (and perhaps unobservable) displacements. Therefore, an observation of an "unfailed" foundation have several possible interpretations:

<u>Foundation Lateral Capacity:</u>

Case A:	Fully plastic section event in the first pile did not occur
Case B:	Fully plastic section events in several piles did not occur
Case C:	Fully plastic section event in one pile did occur

<u>Foundation Axial Capacity:</u>

Case D:	Pullout/plunging event in the first pile did not occur
Case E:	Pullout/plunging events in several piles did not occur
Case F:	Pullout/plunging event in one pile did occur

The calibration process was performed for all platforms for the above cases and the results obtained are given in Table ES-1. The prior distributions of B_{fl} and B_{fa} represent the assumed distribution of the bias in the computed ratios of platform ultimate capacity to loading effects. The posterior of B_{fl} and B_{fa} present the shift in their prior distributions due to combining the effect of computed platform ultimate capacities and loading effects, associated uncertainties in the parameters defining these quantities, and the effect of field observations (platform survived, damaged, or failed during Andrew). The posterior distributions are established by using the Bayesian updating process.

The results in Table ES-1 indicate that the posteriors of bias factors vary slightly for calibration cases A and B, and cases D and E. Case C calibration with the assumed observation of a fully plastic section in one pile did not shift the lateral bias factor, whereas due to Case F with assumed observation of pullout/plunging of one pile shift in the axial bias factor is moderate compared to those due to cases D and E. These results indicate that with a better mix of foundation survival and damage cases, the bias factors may have intermediate values (between A and C or D and F).

Caissons were also included in the study because they offered several observed cases of lateral foundation failures. They provided potential to improve the bias estimate for lateral foundation behavior.

The third set of results presented for cases A, B, C for lateral bias factor (B_{fl}) for the jacket platforms, when the effect of observed damage/failures in caissons are included, shall be considered with caution due to differences in the characteristics and behavior of jackets and caissons. This set of results were determined to demonstrate the likely trend for the jacket bias factor when jacket platforms with foundation damage are included (if considered in later investigations). The third set of combined bias factors are not applicable to the bias in the caissons predicted capacity to loading effect ratio.

Figure ES-2 presents the shift of the prior of B_{fl} due to including the effect of jacket cases and the resulting posterior of B_{fl} for Case A. It shows that the prior mean of 1.0 shifts to the posterior mean values of 1.2 due to each of the platforms ST151K and ST177B and to 1.14 due to platform SS139 (T25). All three jackets combined shift the posterior to 1.32.

Figure ES-3 presents the shift of prior of B_{fa} due to the effect of jacket cases and the resulting posterior of B_{fa} for Case D. It shows that the platform ST177B has maximum influence and platform SS139 (T25) has minimum influence on the shift to the prior. The prior mean of 1.3 shifts to posterior mean values of 1.63 and 1.73 due to individual platform ST177B and due to all three jacket platforms respectively.

CONCLUSIONS AND RECOMMENDATIONS

Geotechnical Aspects

The variation in the pile lateral capacity is determined to be on the order of square root of the variation in the undrained shear strength. In addition, only the lateral p-y springs in the upper zone (up to 60 ft below mudline) get fully mobilized and influence the lateral capacity estimates. The influence of errors in estimates of the undrained shear strength of clay layers (due to unavailability of site-specific soil reports) is more important for the foundation axial capacity estimates than for lateral capacity.

The various effects, which are likely to increase the foundation axial capacity have been identified but the procedures to incorporate their effect for different soil conditions are not fully known. Some recent investigations have shown wide differences in the estimates of increase in the pile axial capacities. The most important effects to include are loading rate and cyclic loading effects, which are known to have compensating effects.

The earlier research discussed in Section 2 has indicated that the API RP 2A, pile axial capacity (t-z) estimates underpredict the true ultimate pile axial capacity. However, in this study no change to the API pile axial capacity characterization was included and the combined effect of the various factors which may lead to increased axial capacities will be reflected in the bias factors.

Instead based on earlier API studies, a higher mean of 1.3 with a COV of 0.3 was considered for the prior of the bias factor (B_{fa}). The increased mean of the prior is considered to provide a reasonable balance between the effects of the loading rate, cyclic degradation, and choice of undrained shear strength of soil.

Capacity Assessment

For both 8-leg platforms, the pullout and plunging of multiple piles were predicted at very low load levels. These load levels are 18 to 33 percent below the best estimate of the maximum Andrew loading. Whereas, the multiple plastic section events were predicted at load levels up to 14 percent higher than the predicted maximum Andrew load level for these platforms, when API static p-y curves were used. The analysis predicted very small pilehead displacements even at multiple fully plastic pile section events and at multiple pile pullout/plunging events in both lateral and axial directions. Considering that neither failure mode was observable in the post-Andrew field inspections, these results indicate that the predictions of the ultimate lateral capacity of pile foundations per API static p-y curves may not be overly conservative, whereas the API t-z curves may underpredict the axial capacity of the jacket foundation system.

In case of ST151K (with no observed damage to the jacket and its foundation) significant differences have been found between the predicted failures to the jacket frames and the field observations following Andrew. It is also noted that the low ultimate capacity estimates may be triggered by pile axial (t-z) capacity estimates, which may be conservative. This is a significant finding suggesting that the ultimate overturning capacity of platforms may be underpredicted in some cases.

It is recommended that additional platforms be investigated to further improve the findings of this study and to further develop the improved guidelines for the ultimate capacity analysis of the steel jacket foundation under extreme storm condition.

Calibration

Bias factors of 1.3 and 1.7 were established for the foundation lateral (B_{fl}) and axial (B_{fa}) computed ratios of ultimate capacity to loading effects (Ru/S) respectively for the steel jacket platforms. This implies that on an average, for the platforms investigated in this project, there is about 30 percent conservatism in the foundation lateral capacity and about 70 percent conservatism in the foundation axial capacity estimates based on the ultimate capacity analysis "recipe" used by the project. These B's are related to the key capacity analysis recipe items followed in this project, thus any major variations in the recipe would influence them.

These estimates are higher than the overall (system) bias factor (B) of 1.2, determined in the Andrew Phase I. The Andrew Phase I bias factor was not applicable to a specific failure mode (jacket structure or foundation) and thus included failure modes of all type. If the effect of the foundation is eliminated to determine the bias factor associated with the superstructure (B_{u}), it is likely that the bias factor will be lower than that obtained in the Andrew Phase I.

These bias factors (B_{fl} and B_{fa}) are based on a limited number of the steel jacket platforms investigated in this project and thus should be viewed as initial estimates and be considered with caution. It is important to note that these bias factors are not applicable to the capacity estimates of an individual platform and that these are for consideration by the API and the regulatory bodies in their further considerations for the guideline and criteria development for platform assessment against extreme storms. These factors may be considered in determination of the average failure probability estimates and in the economic risk and cost-benefit studies for a fleet of platforms. The estimated values for the bias factors are consistent with a trend determined by other investigators.

PMB Engineering is currently executing Andrew JIP — Phase II, which includes all platforms investigated in this study and an additional six steel jacket platforms. In the Phase II work, revised Andrew hindcast data and a revised ultimate capacity analysis recipe are being used, which could affect the findings of this project. Andrew Phase II is using a more detailed reliability evaluation and Bayesian updating procedure, which would result in multiple bias factors applicable to both the jacket structure and its pile foundation.

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

- Increase the sample cases and include searching for recorded foundation failure cases. The platforms with observed foundation damages during earlier hurricanes (e.g., Hilda, Betsy, Camille etc.) may be included to provide useful information to the bias factor based on survival cases only.

- Consider using improved site-specific geotechnical information by obtaining new boreholes for platforms.

- Investigate the differences in foundation design and installation practices among the older and newer platforms, and their likely effect on bias factors. This study considered platforms installed in 1960's.

- Promote more extensive instrumentation and monitoring of existing platforms.

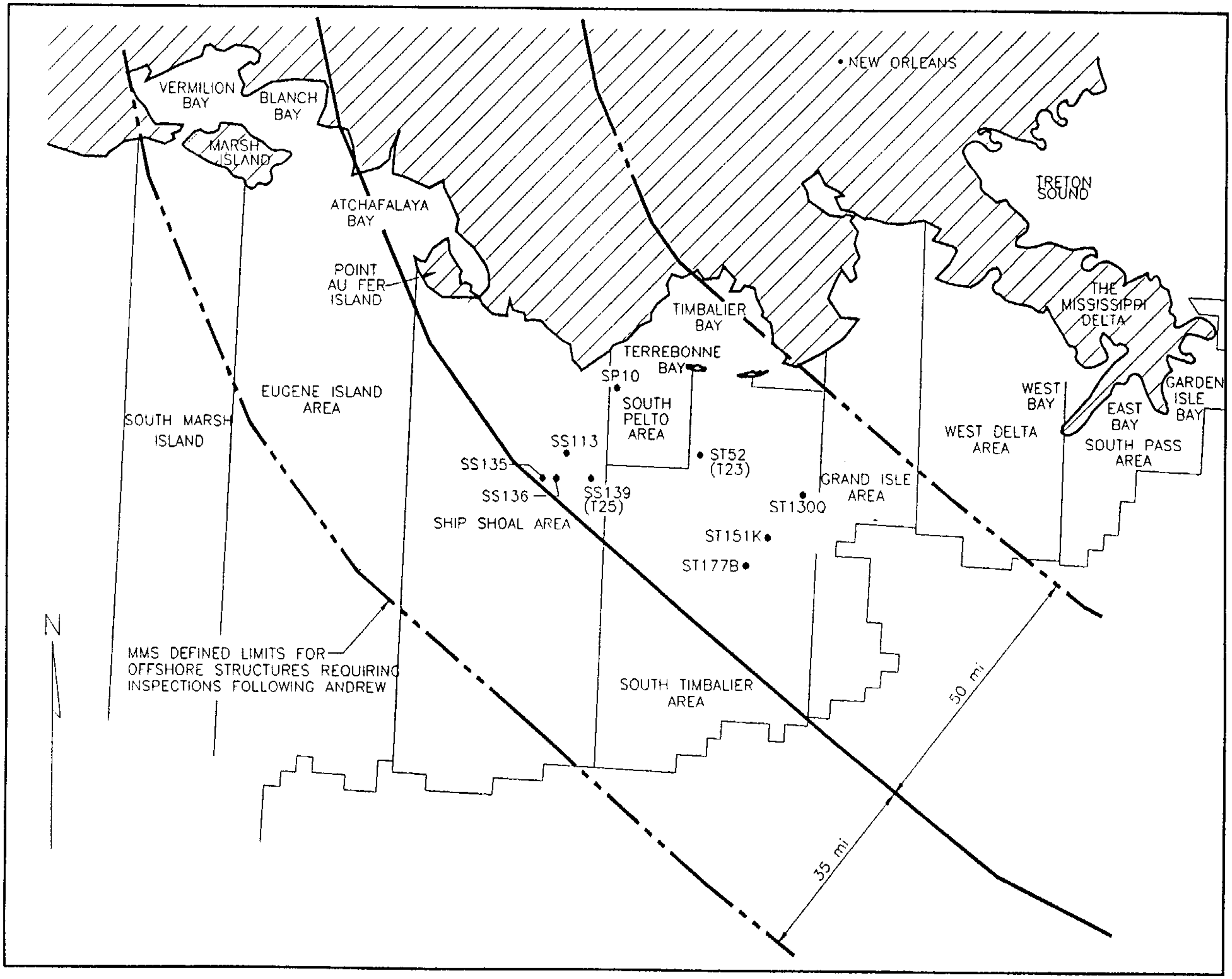
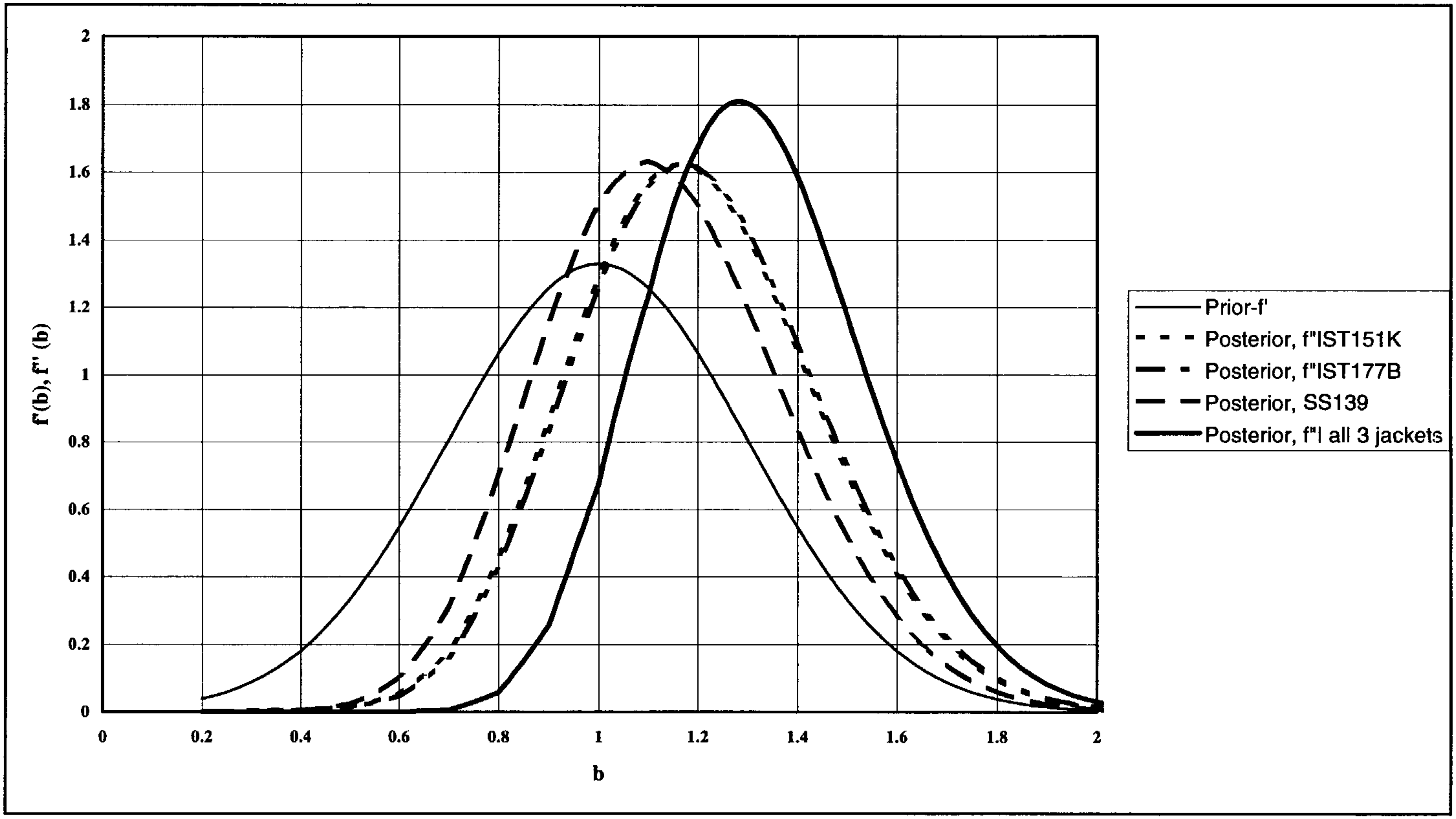
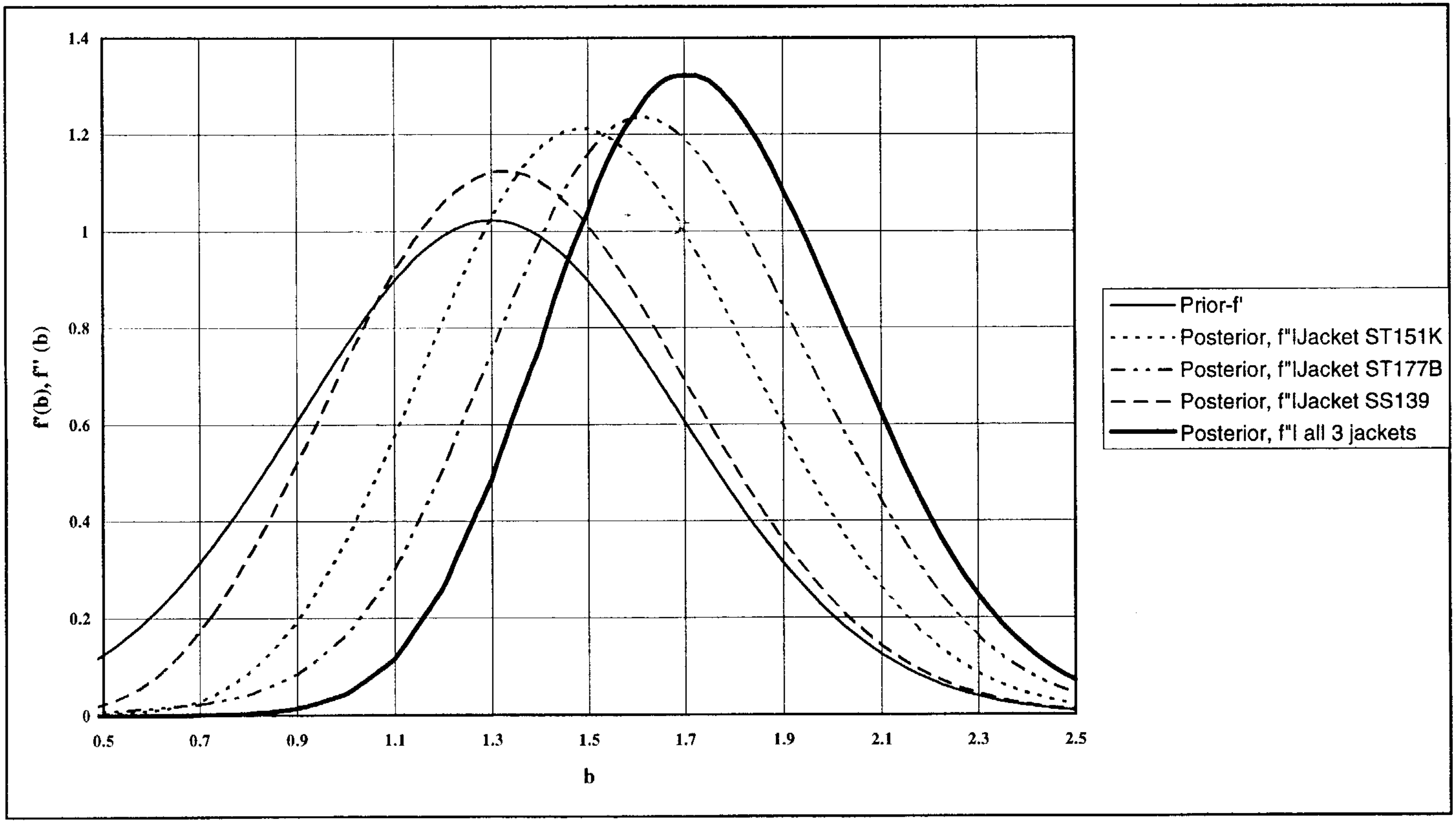


Figure ES-1 Path of Hurricane Andrew with Platforms Used in Calibration



Prior of B_{fl}:	Mean = 1.00, COV = 0.30
Posterior of B_{fl}:	Mean = 1.32, COV = 0.17

Figure ES-2: Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case A



Prior of B_{fa}: **Mean = 1.30, COV = 0.30**

Posterior of B_{fa}: Mean = 1.73, COV = 0.17

Figure ES-3: Posterior Distributions: Axial Capacity Bias Factor (B_{fa}) - Case D

Table ES-1: Summary of Calibration Results

Foundation Lateral Capacity (Bfl):

Calibration Case	Posterior of Bfl - Lateral Capacity (#1)	
	Mean Value of Bfl	COV of Bfl
Case-A: Full Plasticity Event in the First Pile did not Occur:		
3 Jacket Cases Only	1.32	0.17
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	1.09	0.15
Case-B: Full Plasticity Events in Several Piles did not Occur:		
3 Jacket Cases Only	1.26	0.18
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	1.04	0.16
Case-C: Full Plasticity Event in One Pile did Occur:		
3 Jacket Cases Only	1.00	0.16
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	0.91	0.14

Notes: (#1) - Prior Distribution of Bfl Assumed with Mean of 1.0 and COV 0.30

Foundation Axial Capacity (Bfa):

Calibration Case	Posterior of Bfa - Axial Capacity (#2)	
	Mean Value of Bfa	COV of Bfa
Case-D: Pullout/Plunging Event for the First Pile did not Occur:		
3 Jacket Cases	1.73	0.17
Case-E: Pullout/Plunging Events in Several Piles did not Occur:		
3 Jacket Cases	1.66	0.18
Case-F: Pullout/Plunging Event in One Pile did Occur:		
3 Jacket Cases	1.53	0.18

Notes: (#2) - Prior Distribution of Bfa Assumed with Mean of 1.3 and COV 0.30

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 Gulf of Mexico 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 and a joint fracture, to catastrophic damage such as complete collapse of the structure.

An extreme event such as Andrew provided 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 provided a unique opportunity to study offshore structures tested under full scale real conditions.

In October, 1993, PMB completed a joint industry project, "Hurricane Andrew — Effects on Offshore Platforms," (hereafter called Andrew Phase I) and established a bias factor for platform capacity estimate (ratio of predicted platform ultimate capacity to maximum Andrew load). Numerous capacity analyses using CAP (by PMB and one participant) and two other software packages (by two participants) were performed for calibration of the predicted capacity per current industry practice with their actual "observed" behavior. The bias factor was then established by application of structural reliability analysis and the use of a Bayesian updating technique. This bias factor reflects correction to the analytical predictions, so that the analytical results more closely agree with observed results.

The capacity analysis performed in Andrew Phase I indicated a significant number of platforms with predicted failures in pile/soil foundation elements. The failure modes determined were: first yield of pile sections, full plasticity of pile sections, inadequate axial soil capacity leading to pile pullout or plunging. In many cases, the failure mechanisms were formed due to multiple events in the pile/soil foundation system and defined the ultimate capacity estimates.

The bias factor established in Andrew Phase I was an overall (system) correction factor and was not specific to the biases associated with the individual failure modes of the jacket and its foundation. Therefore, to further investigate the bias in the foundation capacity estimates for steel jacket template platforms, API and MMS awarded this study to PMB to specifically evaluate the effects of Andrew on their foundations.

1.2 OBJECTIVES

There were two primary objectives for the API/MMS Andrew Foundation study:

1. **Calibration.** Perform a calibration of procedures for foundation capacity analysis (lateral and axial) for assessing existing platforms. A rigorous method for calibration developed in Andrew Phase I is to be used. The process includes reconciling analytically predicted platform damage and failure with observed field performance during Andrew.

2. **Parameters Influencing Bias Factors.** The various parameters which influence the bias factors are to be identified.

API TG 92-5 has currently finalized a document, "Assessment of Existing Platforms to Demonstrate Fitness For Purpose," which will be issued by API in mid 1995 as a new Section 17 or Section R and will be distributed as a supplement to the API RP 2A, 20th Edition and its LRFD versions, respectively. This document suggests the use of ultimate capacity analysis for assessment of existing platforms in order to identify the critical platforms for possible mitigation measures.

More recent research by Exxon, NGI, and other companies, and results from platform instrumentation programs have indicated that the actual behavior of the pile-soil system subjected to extreme loads (during hurricane events) may significantly differ from the regular design per API RP 2A. Such differences have been determined relatively recently and have not yet become practice. For assessment of existing platforms against overloading from hurricanes, one may consider contribution of such effects in determination of unbiased estimates of ultimate capacity of the foundation system. Thus, a secondary objective of this project was to identify such differences.

A major goal of this project is the calibration of some of the platforms that survived, were damaged or collapsed during recent hurricane Andrew with intent primarily to determine the degree of conservatism in the predicted foundation capacity estimates.

1.3 PLATFORM SELECTION

A significant effort was spent in selection of platform cases, obtaining site-specific geotechnical reports, obtaining information for caissons, and contacting companies for other pertinent information.

The MMS platform failures database was thoroughly investigated in Andrew Phase I and the platform damages were summarized [PMB Engineering, 1993]. This database indicated that only one foundation failure occurred for platform ST172A owned by Samedan Oil Corporation, which leaned 20 degrees with tearing of one jacket leg, 2 deck legs, and pullout of two piles (by 15 ft and 5 ft). This platform with a foundation failure during Andrew was considered to be of significant value to this project. Unfortunately, Samedan declined to provide information for their platform for use in this project.

In lieu of information for this platform, the API Steering Committee and the project team decided to utilize the available information for caissons of which many exhibited foundation failures during Andrew. The expectation was that the caissons would provide a limit to the bias factors established in this project. However, the caissons' effect on the bias factors would be presented separately, to distinguish between individual effects of jackets and caissons.

The platforms were selected from a detailed evaluation using the following available information:

- Structural characteristics and details of platforms
- Damage to the platforms
- Geotechnical information in the vicinity of platforms
- Hindcast information (same as for Andrew Phase I)
- Ultimate capacity and Bayesian updating results from previous analyses (Andrew Phase I results)
- New ultimate capacity analysis done before the first project meeting

The results of this evaluation are summarized in Tables 1-1 and 1-2 for steel jacket platforms and caissons cases respectively. These tables present information for 7 steel jacket platforms and 11 free standing caisson platforms. Figure 1-1 provides locations of platforms and caissons used. The selection process is discussed in more detail in Appendix A.

The following structures were finally selected for detailed investigation in this project:

Steel Jacket Platforms	ST151K ST177B SS139 (T25)
Caissons	SPelto 10 SS113 SS135 SS136 (alternate)

For caissons SS113 and SS136, the site-specific soil information was not available. The final selection of one of these two caissons was dependent upon a review of their capacity analysis results, site-specific hindcast data, and evaluating the likelihood of their providing sufficient information for calibration.

The pile makeups for the steel jacket platforms are given in Appendix A.

1.4 ACKNOWLEDGMENTS

The efforts of the API Technical Advisory Committee for this project were greatly appreciated. The two project meetings and one earlier briefing at an MMS workshop in New Orleans in December 1993 provided much of the interaction during the project. Dr. Jen-Hwa Chen, nominated as Chairman of the project Technical Committee, was contacted more regularly during the project. The API Committee overseeing this project and other technical representatives of companies included the following:

CHEVRON	— Jen-Hwa Chen
DAMES AND MOORE	— Billy Villet
EXXON	— Don Murff
MINERALS MANAGEMENT SERVICE	— Charles Smith
NORWEGIAN GEOTECHNICAL INSTITUTE	— Suzanne Lacasse
SHELL	— John Pelletier
UNOCAL	— Rick Dupin

In particular, the following should be commended for their input:

- **CHEVRON.** Chevron provided several geotechnical reports for use in the project. Chevron also provided a significant number of the platforms (for calibration) during Andrew JIP (also investigated in this project) and additional information for several caissons for use in this JIP, without which the project would not have been possible.

- **EXXON.** Exxon provided technical information during Andrew JIP regarding soil properties that should be used for pushover analyses [Hamilton and Murff, 1995].

- **MOBIL.** Mobil provided information for two caissons damaged during Andrew. The driveability records for both caissons were also provided.

- **MURPHY.** Murphy provided information for five caissons damaged during Andrew. The original design summary reports and information for the redesigned caissons post-Andrew were provided.

- **OCEANWEATHER, INC.** The meteorological and oceanographic site specific hindcast information for Hurricane Andrew provided by Oceanweather, Inc. under an agreement with PMB during Andrew JIP [Oceanweather, 1992].

- **TRUNKLINE.** Trunkline provided geotechnical reports for two platforms during this study. Trunkline also provided data for three of their platforms during Andrew JIP, which were also used in this project.

PMB Engineering Inc. was the prime contractor, providing all project management, analysis (structural and calibration) and reporting. Key PMB staff members for this project were Dr. Dick Litton and Dr. Rajiv Aggarwal. Mr. Dan Dolan and Mr. Frank Puskar of PMB were consulted during this project. In addition, the following provided consulting services to PMB during the project:

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

Dr. Wilson Tang, University of Illinois — input to calibration and geotechnical aspects. Review and comment of results.

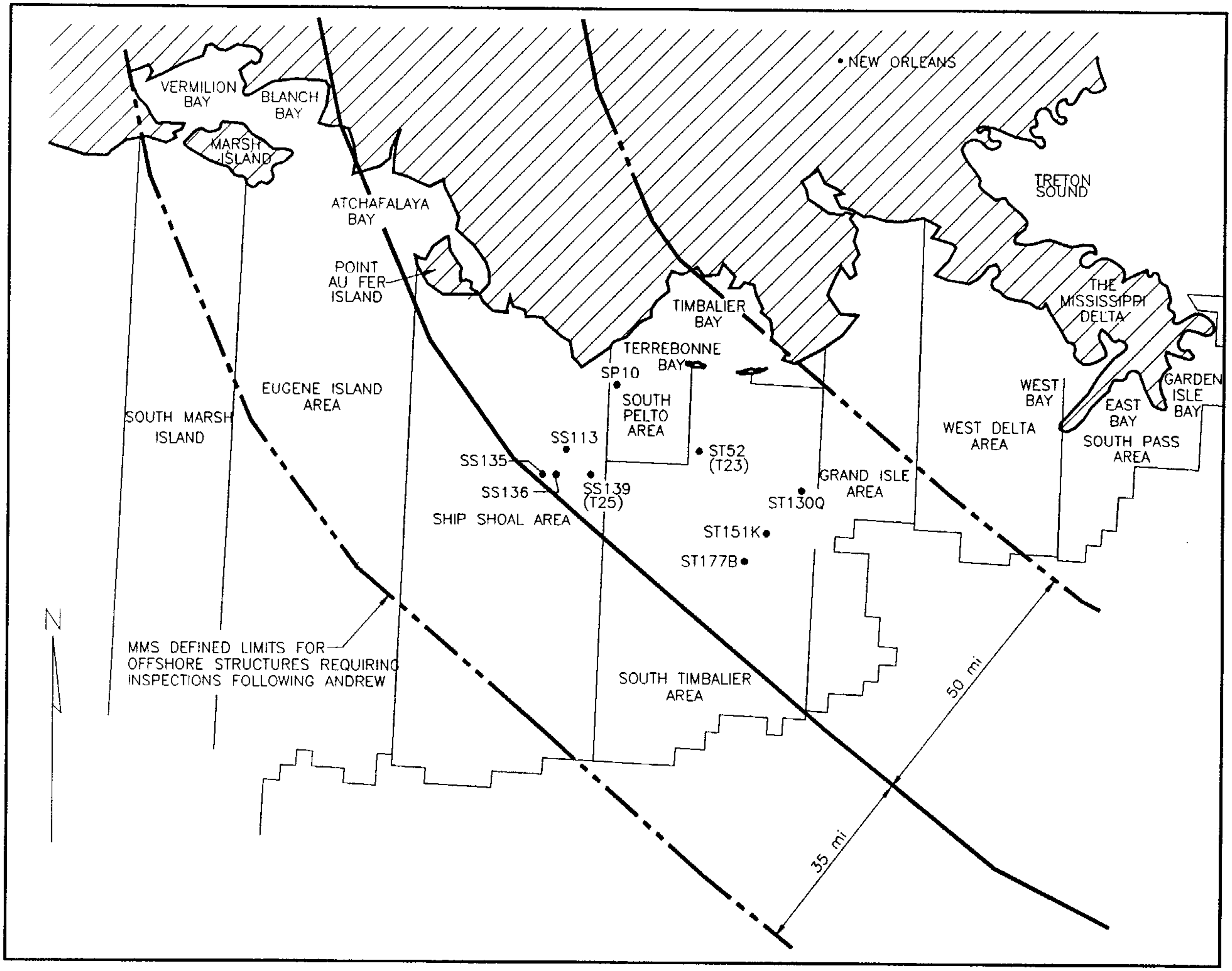


Figure 1-1 Path of Hurricane Andrew with Platforms Used in Calibration

Table 1-1: Features of Steel Jacket Platforms

Platform Identification	Block Number	Year Installed	Water Depth (ft.)	Number of Legs/Piles	Foundation Features		Damage Description (#1)	Availability of Structural Details	Availability of Soil Report	Case Type for Bayesian Updating	Selection for the Foundation JIP
					Pile Diameter (inch)	Penetration Below Seabed (ft.)					
ST 151K	South Timbalier 151	1963	137	8	30 inch	175 ft.	None	Yes	Yes (same block)	Survival case	**YES**
ST130Q	South Timbalier 130	1964	170	4	30 inch	210 ft.	None	Yes	**No** (closer to ST134 borelog)	Survival case	**(#2)**
T23	South Timbalier 52	1969	63	4	36 inch	170 ft.	Tearing of K-joints	Yes	Yes (same block)	Damaged case	**(#3)**
T25	Ship Shoal 139	1969	62	4	36 inch	165 ft.	Tearing of K-joints (Jacket salvaged)	Yes	Yes (same block)	Damaged case	**YES**
ST 177B	South Timbalier 177	1965	142	8	30 inch	187 ft.	K-joints, X-joints (Jacket salvaged)	Yes	Yes (ST189 borelog at about 1 mile)	Damaged case	**YES (#4)**
ST 151H	South Timbalier 151	1964	137	8	30 inch	180 ft.	Rubbled Details not available	Yes	Yes (Same Block)	Collapse case	**(#5)**
ST 130A	South Timbalier 130	1958	180	8	30 inch	?	Leg failure occurred before pile yield	Yes	**No** (farther away from ST134)	Collapse case	**(#2)**

Notes:

#1 Damage observed post hurricane Andrew

#2: Soil report in the same block or in vicinity not available

#3: Preliminary evaluation (presented at the 1st Meeting) showed that it will not give required information

#4: Damaged case with soil borelog at about 1 mile, hence selected

#5: Adequate damage information not available, thus may not provide sufficient information for calibration.

Table 1-2: Features of Free Standing Caisson Platforms

Company	Block Number	Year Installed	Water Depth (#1) (ft.)	Structural Features: Caisson Diameter (inch)	Structural Features: Penetration Below Seabed (ft.)	Damage Description (#2)	Availability of Structural Details	Availability of Soil Report	Case Type for Bayesian Updating	Selection for the Foundation JIP
A	Ship Shoal 99	1961	21	48 inch	59 ft.	Broken at mudline (may be fatigue damage)	Drawings for particular location not available (#3)	Yes (Same Block)	Damaged case (may not be a foundation failure)	(#5)
	Ship Shoal 98	1986	17	72 inch	75 ft.	No damage	Yes	No	Survival case	(#5)
	South Timbalier 52	1977	60	6 ft. to 8 ft. tapered	103 ft.	Severed 5' above mudline	?	Yes (Same Block)	Damaged Case (not a foundation failure)	(#5)
	South Timbalier 51	1982	55	6 ft. to 8 ft. tapered	93 ft.	No damage	Yes	No	Survival case	(#5)
B	South Pelto 9	1985	35	30 inch	195 ft.	Leaning	Yes	No (in vicinity of SPelto 10)	Damaged/Failure case (Foundation failure)	-
	South Pelto 10	1984	35	30 inch	195 ft. (as designed) 180 ft.(as installed)	Leaning 12 degree	Yes	Yes (Same Block)	Failure case (Foundation failure)	Yes
C	Ship Shoal 113	1990	46	48 inch	100 ft.	Toppled over	Yes	Yes (#4) (borelog within 0.2 miles)	Failure case (Foundation failure)	-
	Ship Shoal 113	1988	48	48 inch	100 ft.	Leaning 48 degree	Yes	Yes (#4) (borelog within 0.2 miles)	Failure case (Foundation failure)	-
	Ship Shoal 113	1990	47	48 inch	100 ft.	Leaning 5 degree	Yes	Yes (#4) (borelog within 0.6 miles)	Damaged/Failure case (Foundation failure)	Yes (#6)
	Ship Shoal 135	1983	53 (48' original)	48 inch	100 ft.	Leaning 15 degree	Yes	Yes (borelog within 0.8 miles)	Failure case (Foundation failure)	Yes (#6)
	Ship Shoal 136	1983	50	48 inch	100 ft.	Leaning 30 degree	Yes	Yes (Same well location)	Failure case (Foundation failure)	Yes (#6)

Notes:

#1 Where available, the as-is water depth is mentioned.
#2 Damage observed post hurricane Andrew
#3 Penetration for standard design differ
#4: Same soil report used in design
#5: Failure cases are sought to provide an upper bound to the "bias Factor" distribution based on Jacket ca
#6: To select 2 caissons (out of 3) upon review of the site-specific hindcast data and soil shear strength

Section 2
Geotechnical Aspects

The information taken from the soil reports and borelogs, which is important for characterization of lateral and axial pile capacities, is summarized. This information is used to develop the shear strength profiles for clay stratas. The basic formulations in the API RP 2A with some modifications are used in the static pushover analysis to evaluate platform behavior to collapse and estimate lateral and axial capacities of piles. The procedure for establishing the shear strength is presented below. The special pile/soil behaviors which influence the basic estimates per API RP 2A are discussed.

2.1 SOIL INFORMATION

The soil borelogs for the jacket platforms and caissons analyzed in this project indicate that the soil stratas consist of predominantly very soft to very stiff clays. In some cases, intermittent layers of fine gray sands exist. However, the upper zone, which governs the lateral behavior of piles, ranges in most cases between very soft to firm gray clay.

The most predominant parameter defining foundation capacity in clay is the undrained shear strength (Su). The undrained shear strength values depend on sampling techniques (driven or pushed), types of tests performed in field or in the laboratory, and the degree of disturbance of the soil samples.

The pushed samples are more reliable, but for older platforms driven samples were mostly used for defining soil strength parameters. The UC (unconfined compression) tests on driven samples generally provide low shear strength values as they are significantly affected by soil disturbance.

The borelogs used in this project were taken between 1967 and 1989. The UC tests on driven samples were performed for a majority of cases. McClelland reports used for platforms ST177B and SS139 indicate that a factor of 1.2 was included in development of the interpreted shear strength profile. This was done to compensate for the lower strength obtained for the UC tests performed on the driven wireline samples.

Tests have shown that in general, the shear strength based on unconsolidated undrained pushed (UUP) tests is very close to the shear strength based on miniature vane pushed (MVP) samples. The API pile test data base consists primarily of unconfined compression pushed (UCP) as the reference undrained shear strength. Tang performed statistical evaluation of pile test data taken from various sources and obtained correction factors applicable to other sampling/test procedures. The test results were calibrated to the UCP samples [API PRAC 86-29B, Tang, 1988]. The modeling errors in the undrained shear strength associated with each test type are also reported [API PRAC 89-29; Tang and Gilbert, 1992]. It is indicated that the UCP samples give bias in undrained shear strength with mean of 1.18 and COV of 0.27, compared to the miniature vane driven (MVD) samples.

At the first project meeting, the API committee recommended developing shear strength profiles using various geotechnical information such as stress history, plasticity index, water content etc. It was agreed that using a nominal ratio of S_u/σ_v' can provide a good estimate of shear strenght for normally consolidated clays. Using a suitable strength ratio (0.23 recommended), a shear strength profile can be obtained that is representative of the Direct Simple Shear (DSS) strength. The DSS can be used for analysis of laterally loaded piles since it is well correlated to the vane shear strength which was originally used by Matlock in developing the lateral loading criteria currently specified by API. For axial shaft resistance, it was suggested to use the upper bound shear strength from miniature vane tests.

The shear strength profiles and soil borelog information for the seven blocks studied in this project are given in Appendix C. Figure 1-1 presents block locations for jacket platforms and caissons with respect to the Andrew eye track. Out of these seven blocks, the shear strength profiles available for Ship Shoal blocks 113 and 136 (caisson locations) are based on generalized soil stratigraphy using soil borelogs from adjacent blocks approximately 1.5 miles from the caissons. All other soil information is in the same block or within 1 mile of the platform.

Where sufficient data were available, the following three shear strength profiles are plotted and are included in Appendix C:

- Based on strength ratio (Su/σ_v') of 0.23 and assuming an overconsolidation ratio (OCR) of 1.0
- Miniature Vane tests on undisturbed samples
- Interpreted or design shear strength profile from soil reports

Figure C-1 to C-3 and C-5 to C-6 indicate that in general the interpreted or design shear strength profiles are an average of the undisturbed MV test results. These profiles provide higher values of shear strengths compared to the values based on the strength ratio approach (with an assumed OCR of 1.0). Significant difference is noted in the upper zones between the shear strength profiles based on the strength ratio approach and on the MV results. The shear strengths from the other tests are lower compared to the MV tests due to larger sampling disturbances (see borelogs in Appendix C).

Due to the uncertainty associated with the assumed OCR of 1.0 and unavailability of sufficient information to make its accurate estimate, it was decided to use the shear strength profiles based on the Miniature Vane tests in the upper zone for the base case capacity analysis.

Figure C-4 for South Pelto 10 block indicates that below 60 ft. depth, the shear strength values using the strength ratio approach become higher than those using the MV tests and the interpreted shear strength. In this case, per earlier discussion, below 60 ft. depth the higher values per the strength ratio approach are considered in this study.

From review of the soil reports and borelogs additional observations are made as follows:

- All of the soil reports indicate that Driven samples were obtained.
- ST151 test program (1986) also included some intermittent tests with pushed samples (see Table C-1). The MV test results for the pushed samples provides higher shear strength compared to the driven samples results.
- In case of ST189, for depths below 200 ft., the UC tests provided slightly greater strength compared to MV tests, thus the interpreted shear strength profile (in soil report) was based on the UC tests with a modification factor of 1.2.
- In case of Ship Shoal 139, the UC test data was preferred (per soil report) at increasing depths and the interpreted shear strength profile included a modification factor of 1.2.

Figure 2-1 provides a comparison of shear strength profiles for all platforms and caissons. These are the shear strength profiles used in the analysis.

2.2 LATERAL CAPACITY OF PILES

The lateral capacity of a jacket foundation is composed of the contribution of the capacity of the pile itself (combined axial plus bending) and lateral soil resistance provided by the soils in the upper zones. The effect of variation in the mean value of undrained shear strength of clay is minimized because of this combined contribution of pile and soil (the foundation lateral capacity being proportional to the square root of shear strength). Therefore, spatial variations in the undrained shear strength profiles due to non-availability of site-specific soil data will have less impact on ultimate capacity estimates.

The soil capacity is estimated by the API RP 2A procedures and modeled by p-y curves associated with the distributed, uncoupled, nonlinear springs attached to the pile nodes. API RP 2A presents details for developing both static and cyclic curves. In normal design, the cyclic curves are used to be conservative and represent behavior during design loads. More recent tests have indicated that use of cyclic curves for extreme ultimate load analysis can lead to very conservative capacity predictions [Hamilton and Murff, 1995].

It is noted that the ultimate lateral capacity of foundations involve displacements that are significantly greater than the typical test displacements on which the RP 2A p-y behavior is based. Thus, it is likely that the extreme loading would cause the pile to close any existing gaps and deform virgin soil that has not been previously degraded. This behavior is believed to be the reason for obtaining very conservative results using cyclic curves.

Exxon recently carried out tests on the centrifuge and concluded that for ultimate strength analysis the lateral capacity of piles subjected to cyclic loading should be evaluated based on the static API RP 2A strength [Hamilton and Murff, 1995].

Therefore, in this project the static p-y curves were used for the lateral springs to consider large displacements associated with the Andrew load levels.

Tang obtained the model bias and errors (COV's) associated with the API p-y static and cyclic curves by comparison of measured response of field tests with analytical predictions. The results indicated that the current procedures predict maximum moment response more accurately than pile head displacements for working stress levels. The soft clay criterion was identified to provide the best p-y curve for predicting lateral response of piles in clay. Tang recommends the correction factors (biases) for the lateral pile response (maximum bending moment) associated with the static loading p-y curves for the clay case as 0.92 (fixed pile head case) and 1.08 (free pile head case). The corresponding COV's are recommended as 0.20 (fixed pile head case) and 0.09 (free pile head case) [API PRAC 87-92; Tang and Gilbert, 1990]. The sensitivity analyses have indicated that among the soil parameters, the response is most sensitive to the undrained shear strength and less sensitive to the unit weight of soil.

Based on the above, the prior distribution of the bias factor associated with the lateral capacity estimates of a jacket foundation using the API static p-y curves, is assumed for this study to have a mean of 1.0 and a COV of 0.3.

2.3 AXIAL CAPACITY OF PILES

A significant amount of published work is available on axial capacity/behavior of offshore tubular piles in clay. The pile load test results have been compared with the predictions per the methods given in the RP 2A. The following special effects have been identified in the literature for differences in the pile capacity estimates per RP 2A:

- Loading rate or strain rate effect
- Cyclic loading
- Reconsolidation (time effect)
- Compressibility (pile length effect)

In addition, pile aging effects have also been mentioned by Pelletier et al [1993]. Pile aging effects may occur in some cases at constant effective stress due to long-term rearrangement of soil particles after occurrence of soil damage (e.g., due to pile driving or cyclic loading). This effect may lead to an increase in soil strength and modulus and is under study by API investigation.

The offshore piles are subjected to a combination of static and cyclic loads. API combined the first two effects listed above under "cyclic loading effects" (some other works discuss these two effects under "dynamic loading effects") and they can have potentially compensating effects on pile axial capacity:

- High rate of loading (during hurricane condition) increases capacity and stiffness
- Repeated loading results in decreased capacity and stiffness

Loading rate: Cohesive soils respond in a viscous manner to high rate of loading, i.e., their strength depends upon the rate at which they are sheared. These rate effects are greatest in soft plastic clays with high liquidity indices. The resultant effect on capacity is difficult to quantify as it depends upon pile properties, soil characteristics, and loading. The RP 2A Commentary discusses these but no explicit procedures for computation are provided.

Some investigators have suggested that the viscous rate effects around the pile shaft typically cause a capacity increase of 5 to 20 percent per log cycle increase in loading rate, referenced to initial loading rate, with the greater increase typically occurring for softer and more highly plastic clays. Bea et al [1984] mention that in the absence of soil strength degradation due to load cycling, axial capacities of piles in clay that are loaded by storm waves could be 30 to 80 percent greater than capacities calculated using static design procedures.

Using an analytical model, Dunnavant et al [1990] obtained predictions of peak soil resistance during high-intensity wave loading 35 percent greater than peak static resistances for typical Gulf of Mexico clays. The 35 percent increase was obtained for a case with low cyclic loads (4 to 6 percent of static) and considering no cyclic degradation of axial capacity. For cases with high cyclic loads, the predicted rate effect varied with increases noted up to 21 percent.

For a highly plastic clay, loaded in one-quarter of one wave-period, the strain rate effects can double the undrained shear strength. Recent NGI tests on medium plastic clay also confirms this [Lacasse, 1994]. Some other test results have shown that the "rate of loading effect" could result in axial capacity estimates as high as twice the static capacity [Briaud et al, 1984].

In their recent study on normally consolidated clay, Dutt et al [1992] concluded that strain rate effects were within the range of other published data, with shear strength increasing at the rate of 11 percent per log cycle of load rate.

Cyclic loading: The effect of cyclic loading depends on the overconsolidation of the clay and on the ratio of cyclic to static loads. In the case of normally consolidated clay the effect may be small and will increase with increasing overconsolidation ratio (OCR).

Dutt et al. [1992] found that cyclic strength for one-way loading was significantly greater than the static shear strength. Karlsund and Nadim [1990] discuss the effect of variation in cyclic load histories during the storm for different soil elements along the pile. They determined that two-way cyclic loading at the pile top is more critical than one-way cyclic and can lead to a significant reduction of pile capacity.

Reconsolidation: The time-consolidation effect in the Gulf of Mexico clay may be slow for the platforms studied in this project, which are located in relatively shallow water depths. The platforms investigated in this project are older structures with minimum topside facilities and thus have low static load levels.

Compressibility: This is also known as "pile length effect" or "pile flexibility effect". The stiffness of a pile relative to the soil in which it is embedded can influence its cyclic behavior. The amount of skin friction degradation during cyclic loading can depend substantially on the stiffness of pile relative to that of the surrounding soil [Dunnavant et al, 1990]. Longer, more flexible piles may have somewhat lower capacities due to cyclic loading or softening than estimated per the conventional methods.

Semple and Ridgen [1984] suggest that the pile length effects should be considered when the length/diameter (L/D) ratio is greater than 50. Lacasse [1994] mentioned that the uncertainty (bias) on this factor may be around 15 to 20 percent.

The interaction of these effects is complex, and it is the combined effect that is important for this study. The recommended factors of safety contained in RP 2A implicitly recognize these effects, and the importance of each effect for a particular platform will vary. If any of these effects are explicitly taken into account, then the other effects should also be explicitly taken into account as well, to be consistent [Pelletier et al, 1993].

The above effects were discussed with the API Technical Advisory Committee at the first project meeting and the following assumptions were recommended by the committee for this project:

- Consider an average of 10 percent increase in the axial capacity of pile from strain rate effects
- Cyclic degradation is typically low for the Gulf of Mexico clays and it was recommended to refer to cyclic degradation estimations per Dutt et al [1992]
- Pile flexibility effects are partly accounted for in this capacity analysis (CAP models), with detailed modeling of the pile/soil system
- Ignore pile aging effect in this project
- For silty soils, the API formulation is valid for a plasticity index as low as 30 percent

As the project progressed, it was considered that due to large uncertainty in estimates of the contributions of various effects discussed above to the ultimate axial capacity, there will be no attempt to interpret the effect of individual factors on the pile capacity analysis recipe. Therefore, in this project no change to the pile axial (t-z curve) capacity characterization was included and the combined effect of these various effects will be reflected in the bias factors.

Tang investigated biases associated with the API RP 2A pile axial capacity prediction models by considering the above special effects as modeling errors, in a probabilistic model [API PRAC 86-29B; Tang, 1988]. The variations of individual parameters were assessed using available information and sensitivity of pile capacity due to change in the parameters was investigated. The overall bias associated with RP 2A, 16th edition procedure was estimated to range from 1.3 to 3.7 and the overall error (COV) as 32 to 53 percent depending on how the undrained shear strength was determined at the given site. Note that a major contributor to the bias is the loading rate effect which alone has a mean bias of 1.56. These conclusions were based in part on an analysis of a large data base of pile load tests (on smaller diameter piles) compiled by Olson [1984] and additional correction factors were introduced to extend them for predicting pile performance of offshore platforms. These estimates were based on the most heavily loaded piles in a offshore platform.

Tang noted that the model errors will reduce with new capacity prediction models in 17th edition of RP 2A, and the overall bias is more likely to be 1.5 to 3.0 with associated overall error as 30 to 40 percent [Tang et al, 1990].

After careful consideration of the preceding discussion, the prior distribution of the overall bias factor in pile axial capacity is assumed for this study to have a mean of 1.3 and a COV of 0.3. We believe that these values provide a reasonable balance between the effects of loading rate, cyclic degradation and choice of undrained shear strength of soil.

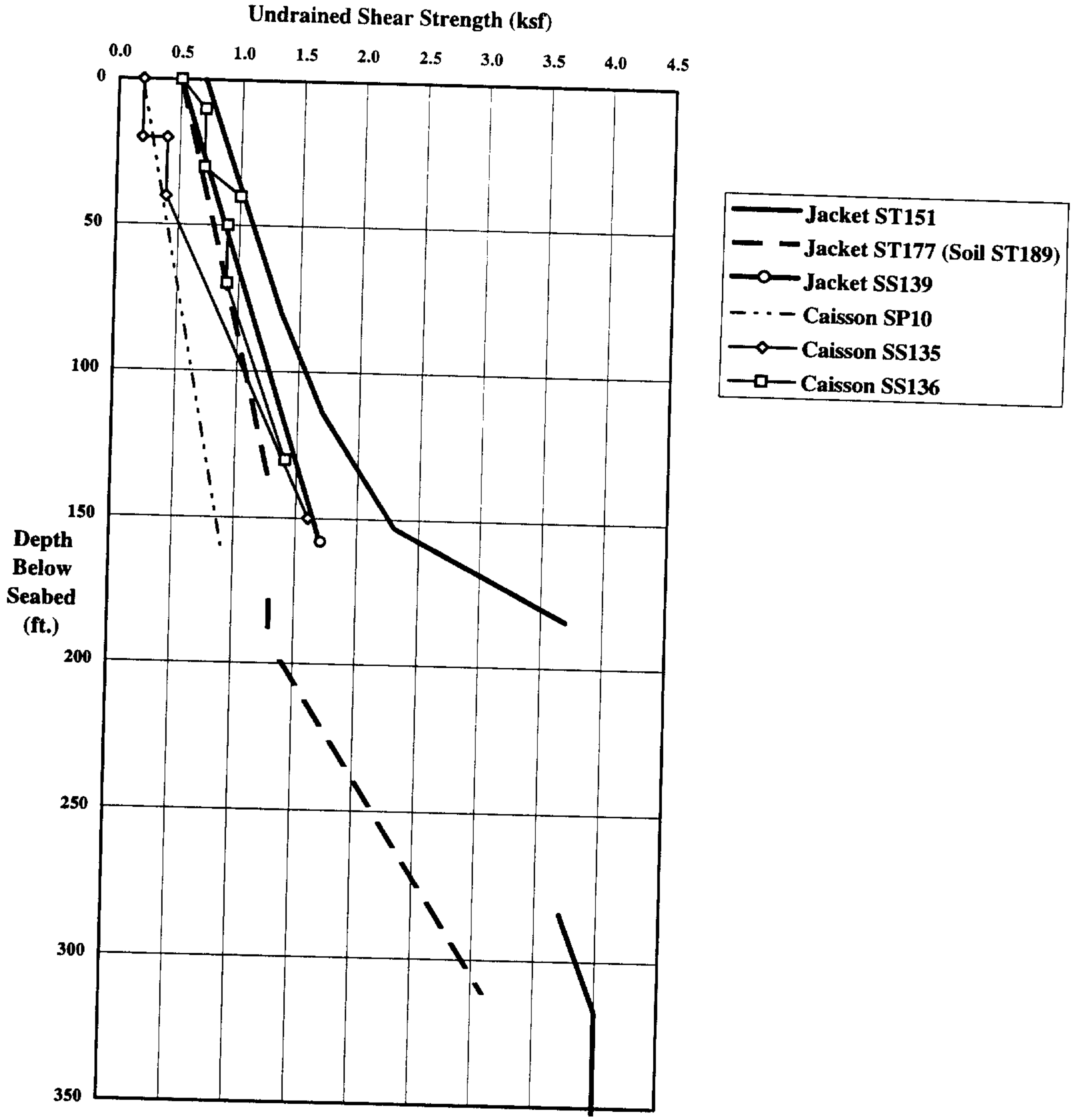


Figure 2-1: Undrained Shear Strength Profiles

Section 3
Capacity Analysis

3.1 APPROACH

The capacity assessments consisted of explicit nonlinear structural analysis. Platform loads and resistances 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 based upon their likely contribution to this project were discussed in Section 1.3. Three steel jacket platforms and three caisson platforms were evaluated, which included platforms that survived, were damaged or collapsed during Andrew.

A static pushover to determine the platform capacity for use in the calibration was performed for each platform using the PMB computer code CAP (Capacity Analysis Program). The static pushover is the typical approach used by the industry to determine the maximum lateral load carrying capacity of offshore platforms. This load can then be related to a specific wave height that can cause platform failure.

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 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 [API, 1995; Puskar et al., 1994; Bea et al., 1988; Lloyd and Clawson, 1983].

The goal of this task was to establish the lateral load levels associated with the predicted failures of individual and multiple elements, which would define the platform ultimate capacity associated with a particular failure mode and the analysis recipe (for both loading and element capacity characterization). Two failure modes (mechanisms), pile yielding/hinging and pile pullout/plunging, defining foundation capacity of platforms against lateral and axial loads, respectively, are investigated in this study.

Three analyses are performed for each steel jacket platform to obtain uncoupled estimates of the lateral load which causes first element events and multiple events (defining foundation capacity) using the following models:

- Base case analysis: Nonlinear jacket and foundation model — to determine critical failure mode for best estimate deterministic model.

- Case 2 analysis: Linear jacket and nonlinear foundation model — to determine critical foundation failure mode for best estimate deterministic model. The analysis may be repeated to suppress one foundation failure mode type if both lateral and axial modes are predicted.

- Case 3 analysis: Case 2 model with one foundation failure mode type (pile pullout/plunging or pile yielding/plastification) suppressed — to determine capacity in the alternate foundation failure mode.

The Case 2 and Case 3 analyses were to provide estimates of load levels corresponding to events in the pile/soil system, by elimination of failures predicted in the jacket elements in the base case analysis and also one type of foundation failure mode (lateral or axial). Thus, the likely effect of uncertainties associated with the characterization of jacket element strength and stiffness are eliminated and uncoupled estimates of foundation failure modes are obtained.

In the case of caissons with only one failure mode (yielding/hinging of the caisson), the base case analysis is sufficient. However, additional analyses with increased values of undrained shear strength and with elimination of thin sand strata were also performed to determine their sensitivity to these aspects.

3.2 LOAD AND RESISTANCE RECIPE

The load and resistance recipe used for the capacity analysis was based primarily upon the API RP 2A 20th edition, with several modifications as required for this project. The basic recipe followed is the same as for Andrew Phase I. 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 in Andrew Phase I.

Key parameters of the recipe (loading and structural) identified in Andrew Phase I 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 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 for X, diagonal and K bracing 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 in 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).

- **Grouted Joints.** The API RP 2A equations for ungrouted joints were used with an equivalent thickness for the leg based upon the strength contributions from the leg, grout and pile [UEG, 1983].

- **Wave Loads on the Deck.** In cases where waves impact the deck, the project used the simplified procedure developed by API Task Group 92-5 [API, 1995].

- **Ungrouted Joints.** The "mean" capacity estimates of joints (K and X joints) used were based upon formulations given in the literature. These estimates of joint capacities are higher than those per API RP 2A formulas used in Andrew Phase I. Where joint capacities governed over the brace capacities, the joints were modeled to fail in a brittle mode within the gap between the K-braces. Such failures (due to Andrew) were noted from post-Andrew inspections of platforms ST177B and SS139.

At the First Project Meeting held on March 22, 1994, various recipe issues especially applicable to the pile/soil system were discussed and agreed with the API Committee. These items were further investigated and discussed in Section 2.

- **Soil Shear Strength:** Shear strength profiles will be developed using a consistent approach for all platforms (see Section 2.1).

  In the case of driven samples, a modification factor of 1.2 will be used to account for the effect of disturbance. With pushed samples, no modification factor will be used.

- **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 and

Murff, 1995], indicate that for pushover type analysis, the static lateral soil strength provide a better ultimate capacity prediction (see Section 2.2 for further details). Therefore, static p-y soil strength (as defined by API RP 2A) was used for all of the analysis.

- **Axial t-z Springs:** Pile flexibility effects are explicitly accounted for in the capacity analysis using CAP, which handles the drop in the skin friction at large pile displacements. Axial soil strength as defined by t-z spring formulation per API RP 2A was used for all analyses, with no reduction in the capacity as shown in Figure 3-1.

  The contributions of loading rate, cyclic loading, reconsolidation and aging effects were not included in t-z modeling (refer Section 2.3).

3.3 ANALYSIS MODELS

Figure 3-1 shows the nonlinear computer model of an 8-leg steel jacket 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 elements is shown. The model included the following:

- **Deck** — Typical linear beam elements since no inelastic response is anticipated. The deck framing was simplified for the analysis. The deck legs were modeled as nonlinear beam-column elements.

- **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 Andrew JIP participants involved in a series of confidential laboratory joint tests.

- **Soils** — Nonlinear p-y, t-z and q-z springs to model pile/soil behavior.

Figure 3-2 shows the nonlinear computer model for a free standing caisson. The model consisted of a fully coupled nonlinear pile-soil system. The caisson and its foundation were divided into a number of elements as shown and were modeled as nonlinear beam-column

Table 3-2: Capacity Analysis Results for Caisson Cases

Platform	Platform Characteristics		Analysis Case	Expected Maximum Hindcast Base Shear	Base Shear (Kips) from Static Pushover Analysis			Displacement		Ratio	System Factor
	Configuration	Water Depth/ Year Installed		(kips)	at First yield of Ist Section (#1) (kips)	at Fully Plastic Ist Section (kips)	at Ultimate Capacity (#2) (kips)	at Deck Level (ft.)	at Seabed level (ft.)	Ult. Cap./ Andrew BS	Ult. Cap./ BS at First inelastic event (#2) / (#1)
SPelto 10	30" dia. 180' penetration 12 degree lean	35 ft. 1985	**Base Case**	**65**	38 at 35' below	**44**	48	5.63	1.2	**0.74**	**1.26**
			Su increased by 100 %	65	54	69	-	-	-	-	-
SS113	48" dia. 100' penetration 5 degree leaning Max. t = 1.75"	47 ft. 1990	**Base Case**	**138**	186 in 1.75" section	**214**	235	7.3	1.67	**1.70**	**1.26**
SS135	48" dia. 95' penetration 15 degree leaning Max. t = 2"	53 ft. 1983	**Base Case**	**132**	119 at 40'-50' below	**128** in 1" section	148	7.2	-	**1.12**	**1.24**
			Su increased 50 %	132	129	142	165	-	-	**1.25**	**1.28**
			Su increased 100%	132	138	154	178	-	-	**1.35**	**1.29**
SS136	48" dia. 100' penetration 30 degree leaning	50 ft. 1983	**Base Case (upper bound)**	**126**	173	197	224	7.24	1.96	**1.78**	**1.29**
			Ignored Sand layer	126	150	153	181	6.78	2.05	**1.44**	**1.21**

Table 3-1: Capacity Analysis Results for Steel Jacket Platform Cases

Platform/	Platform Characteristics: Configuration	Platform Characteristics: Water Depth/ Year Installed (ft.)	Wave Approach Direction from Platform North	Analysis Case	Expected Maximum Hindcast Max. Base Shear (Sa)	Base Shear (at First Failures) from Static Pushover Analysis: Brace/ Joint Event (Rl1) (kips)	Pile First Yield Event (Rl2) (kips)	Pile Plasticity Event (Rl3) (kips)	Pile pullout/ Plunging (Rl4) (kips)	Base Shear (at multiple failures) from Static Pushover Analysis: Brace/ Joint Events (Rs) (kips)	Pile Plasticity Events (Rfl) (kips)	Pile pullout/ Plunging (Rfa) (kips)	Pushover Analysis Ultimate Capacity (Rus, Rufa, or Rufl) (kips)	Platform Collapse Mode	Ratio of Ult. Cap./ Andrew Base Shear = (Ru/Sa)	System Factor = (Ru/Rl) (#1)	System Factor (Pile-Lateral Capacity) = (Rfl/Rl3)	System Factor (Pile-Axial Capacity) = (Rfa/Rl4)
ST 151 K	8 Leg - Grouted Double Battered K Braced 30" dia. piles 175' penetration	137 ft. 1963	Diagonal 301.4 deg.	Base Case (Nonlinear Jacket)	4,860	3,370	3,470	-	3,350	4,000	-	3,930	4,000	K-joints overstress, Pile yielding and pullout	0.82	1.19	-	-
				Case-2 (Linear Jacket)	4,860	-	3,750	-	3,330	-	-	4,000	4,070	Pullout of 3 piles, fully plastic events at two sections for each pile	0.84	1.22	-	1.20
				Case-3 (Linear Jacket with Pile Pullout/Plunging Suppressed)	4,860	-	3,600	4,340	-	-	5,000	-	5,000	All piles with fully plastic event at 40' below mudline	1.03	1.39	1.15	-
ST 177B	8 Leg - Grouted Double Battered K Braced 30" dia. piles 187' penetration	142 ft. 1965	Diagonal 302.6 deg.	Base Case (Nonlinear Jacket)	4,460	2,850	3,070	-	2,200	3,180	-	-	3,200	X-braces, K-joint overstress, Pile yielding and pullout	0.72	1.45	-	-
				Case-2 (Linear Jacket)	4,460	-	3,200	-	2,300	-	-	2,970	3,200	Pullout of 3 piles and Plunging of 3 piles	0.72	1.39	-	1.29
				Case-3 (Linear Jacket with Pile Pullout/Plunging Suppressed)	4,460	-	3,270	4,330	-	-	5,100	-	5,100	All piles with fully plastic sections at two levels	1.14	1.56	1.18	-
SS139 (T25)	4 Leg - Grouted Double Battered K Braced 36" dia. piles 165' penetration	62 ft. 1969	Orthogonal 269.7 deg.	Base Case (Nonlinear jacket)	1,770	1,600	-	-	-	-	-	-	1,600	K-joints events and pile yielding	0.90	1.00	-	-
				Case-2 (Linear Jacket)	1,770	-	1,800	2,100	-	-	2,400	-	2,410	All piles with fully plastic sections	1.36	1.34	1.14	-
				Case-3 (Linear Jacket with Pile Yielding/Hinging Suppressed)	1,770	-	-	-	2,500	-	-	2,500	2,500	Two piles plunge	1.41	1.00	-	1.00

Notes: (#1): Rl the lesser of Rl1, Rl2, and Rl4.

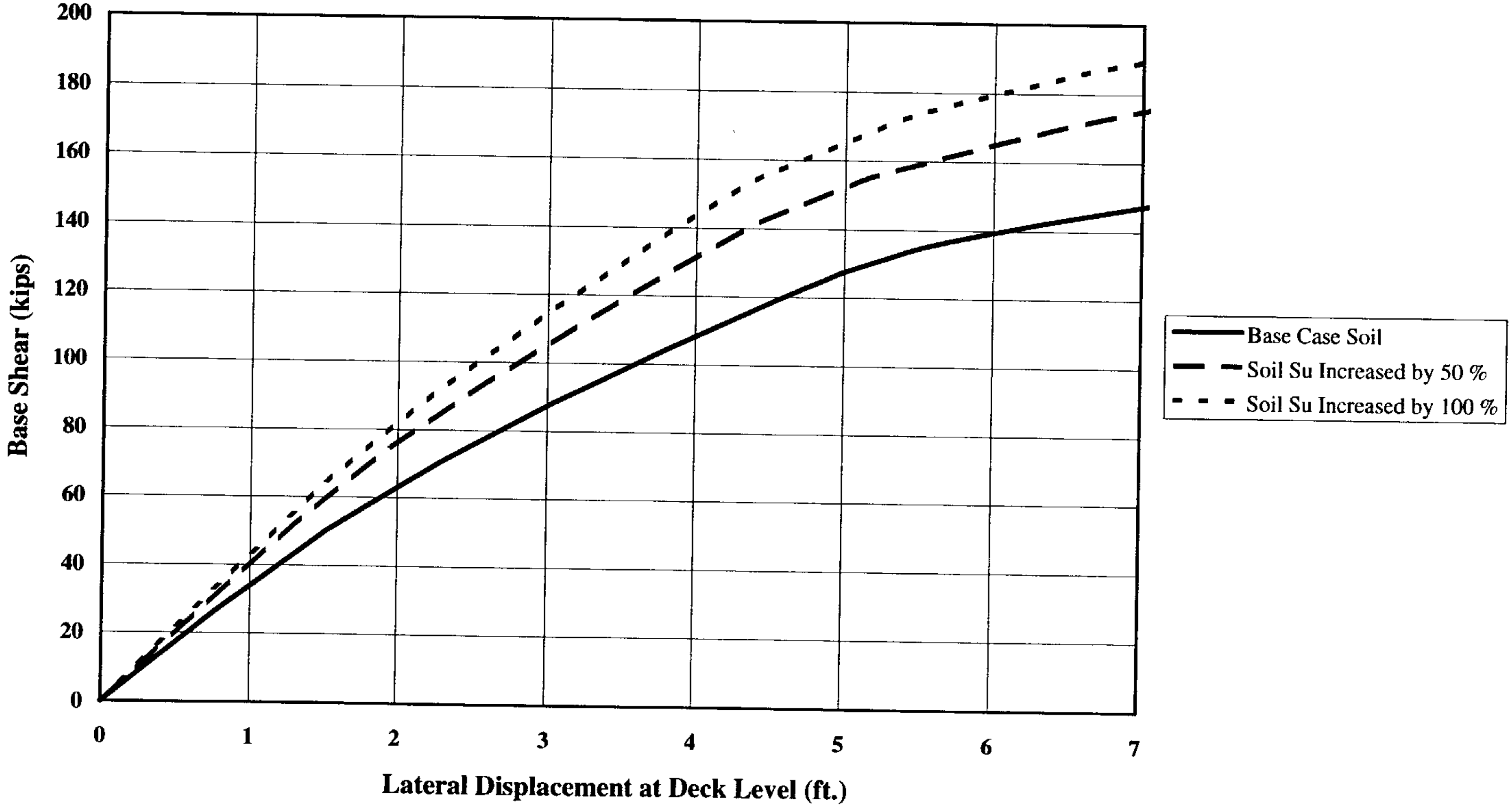


Figure 3-12: Pushover Analysis Results - Caisson SS135

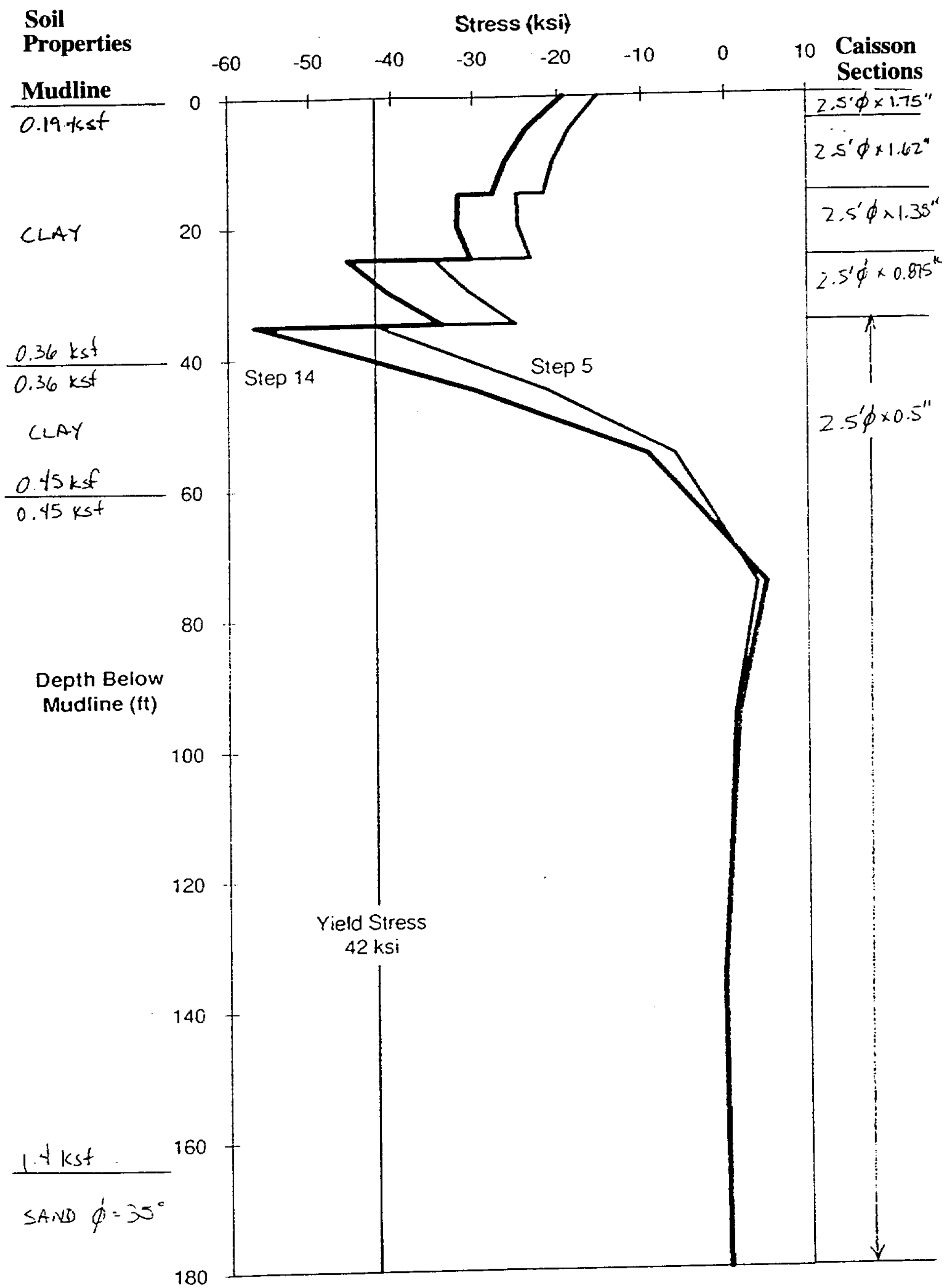


Figure 3-11: Pushover Analysis Results - Bending Stresses - Caisson Spelto 10

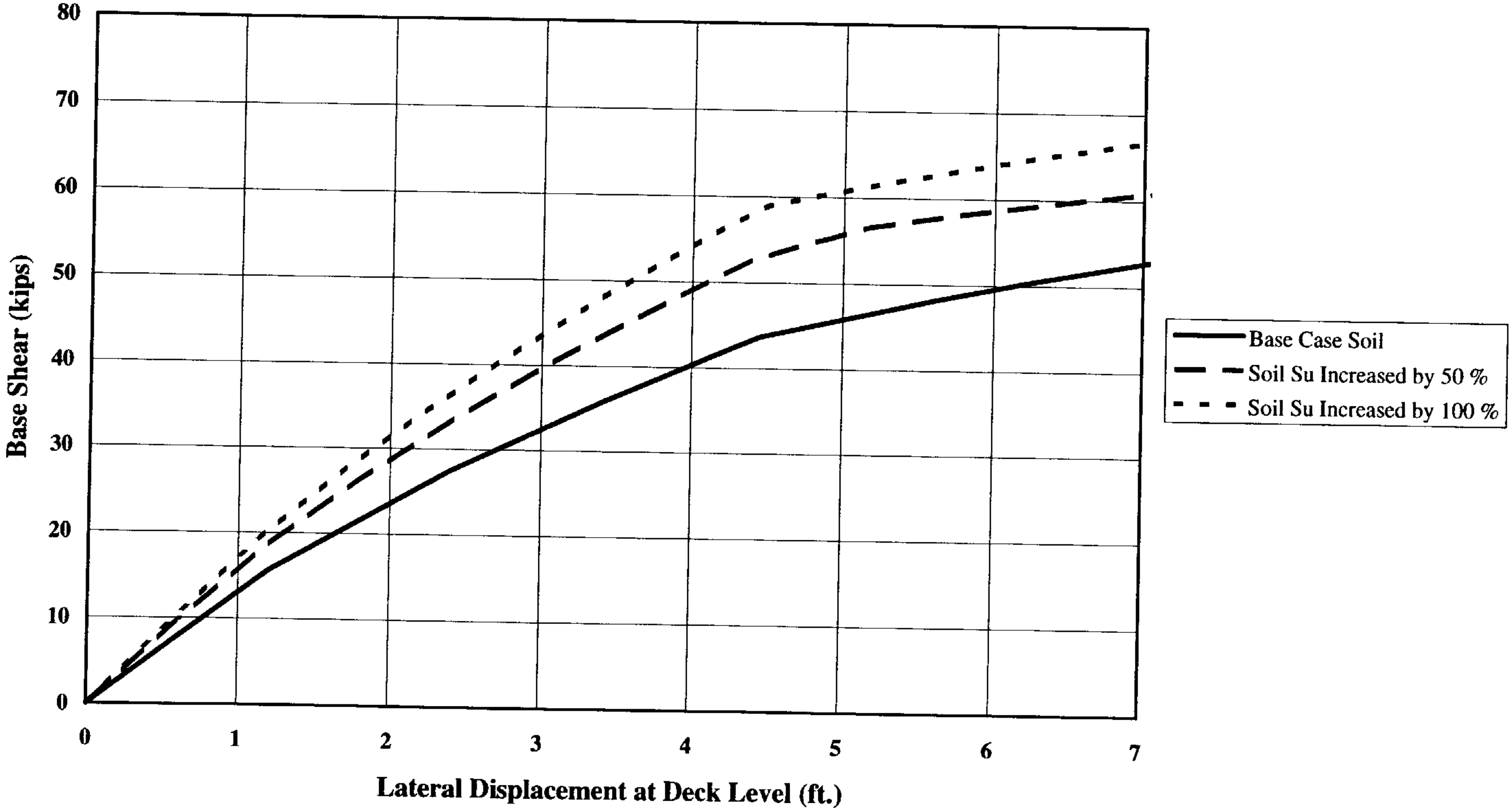


Figure 3-10: Pushover Analysis Results - Caisson SPelto 10

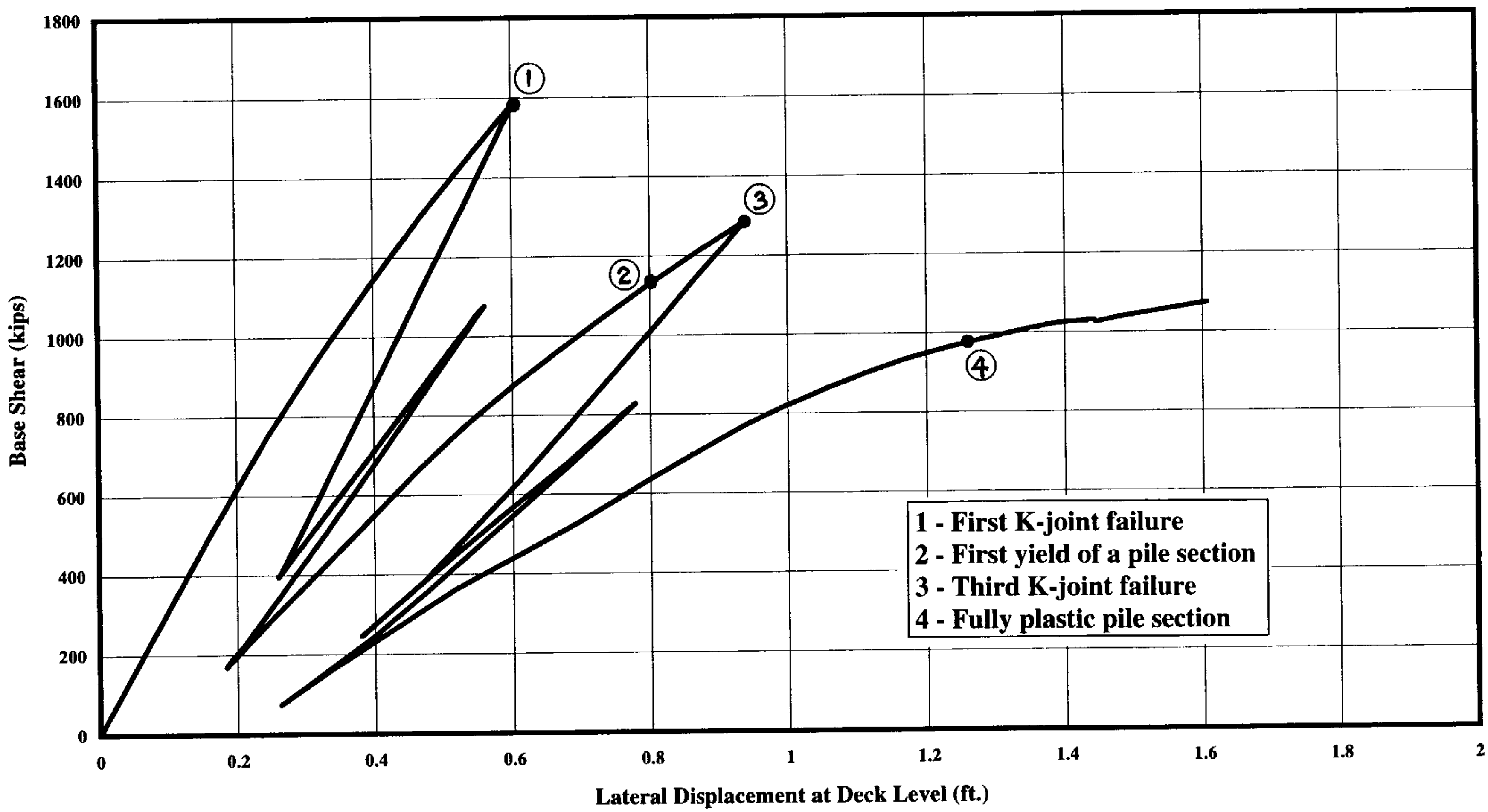
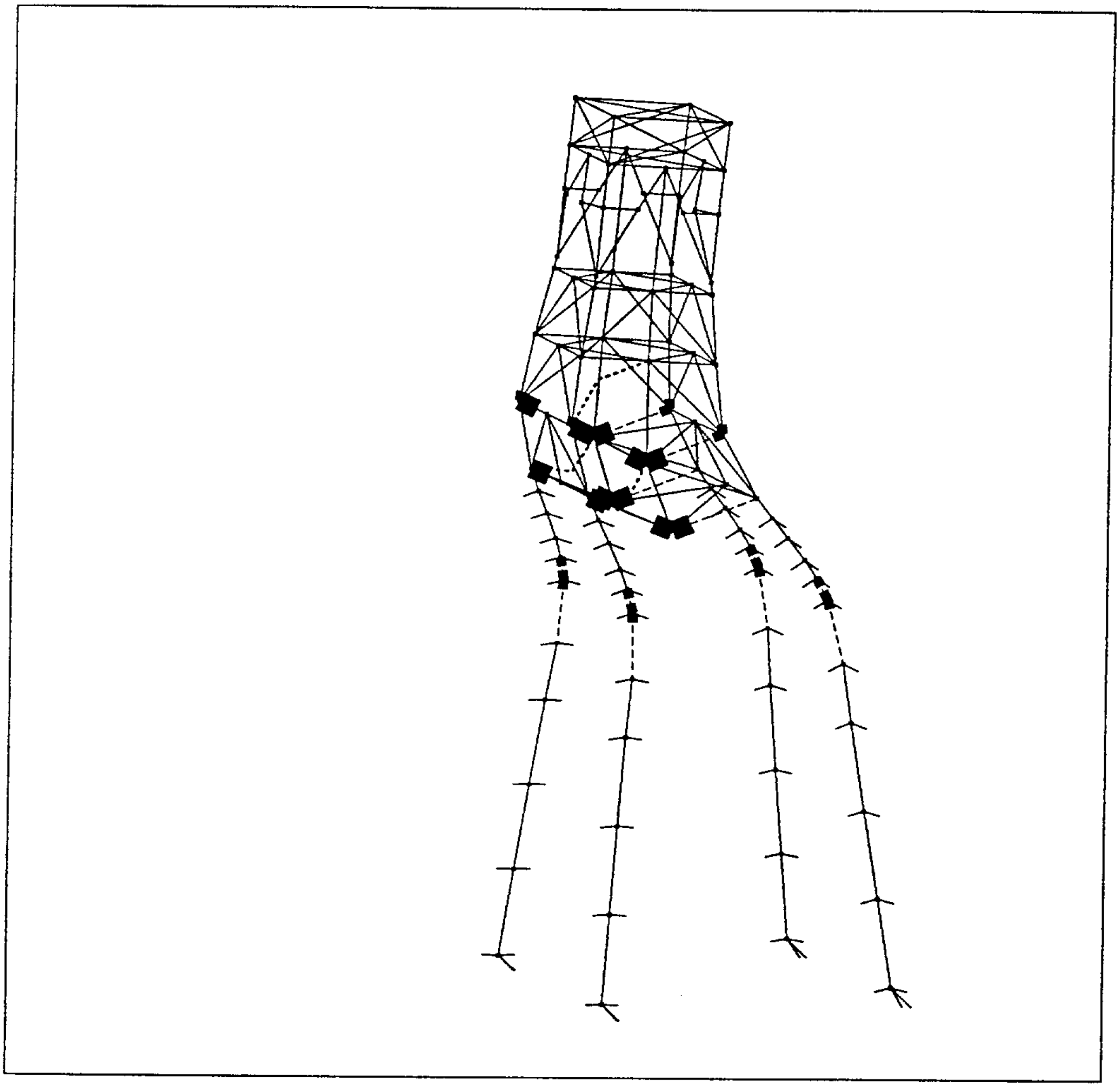


Figure 3-9: Pushover Analysis Results - Jacket Platform SS139



CAP

Inelastic Events Legend

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

Figure 3-8: Pushover Analysis Failure Events - Jacket Platform SS139

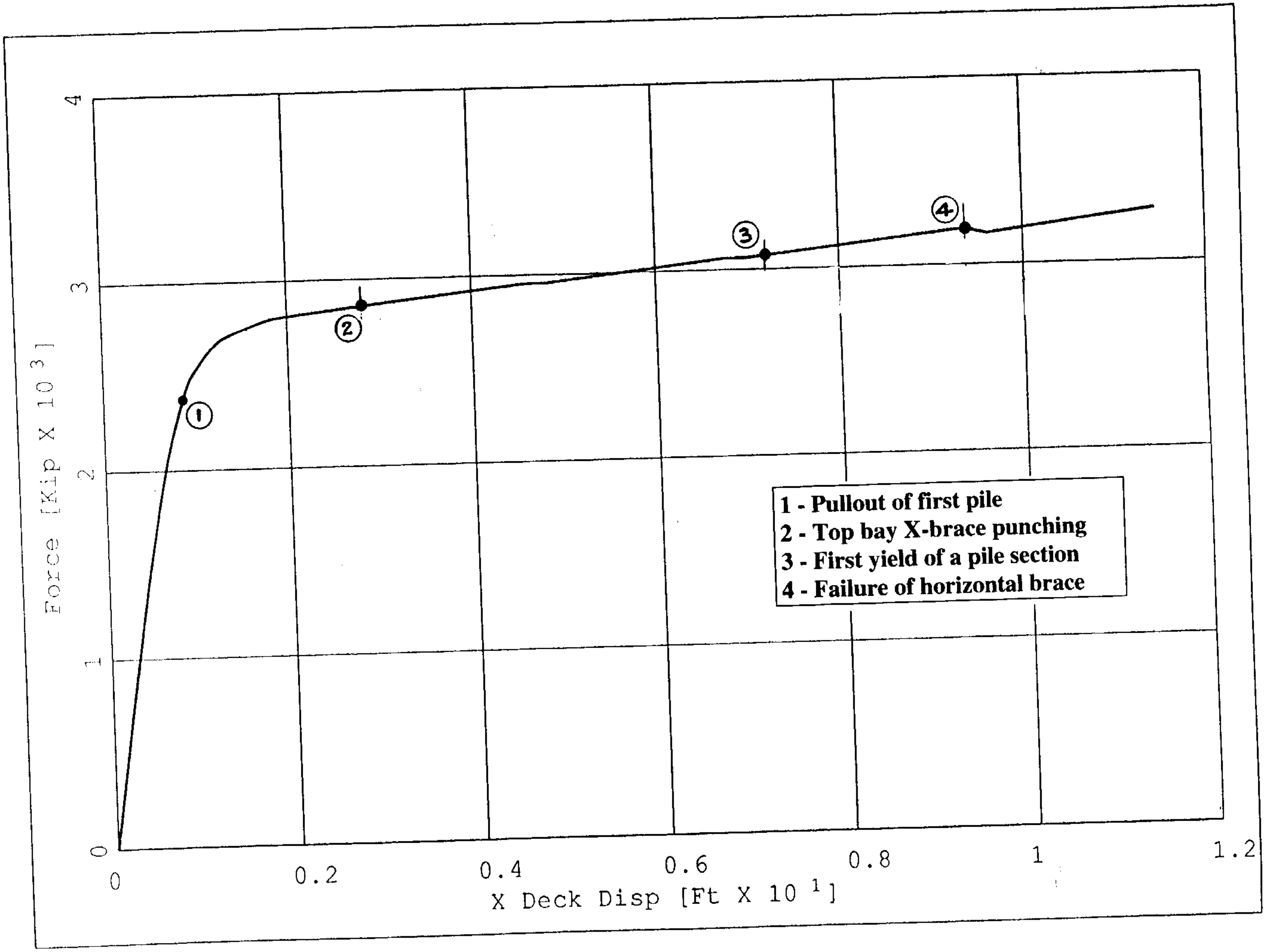
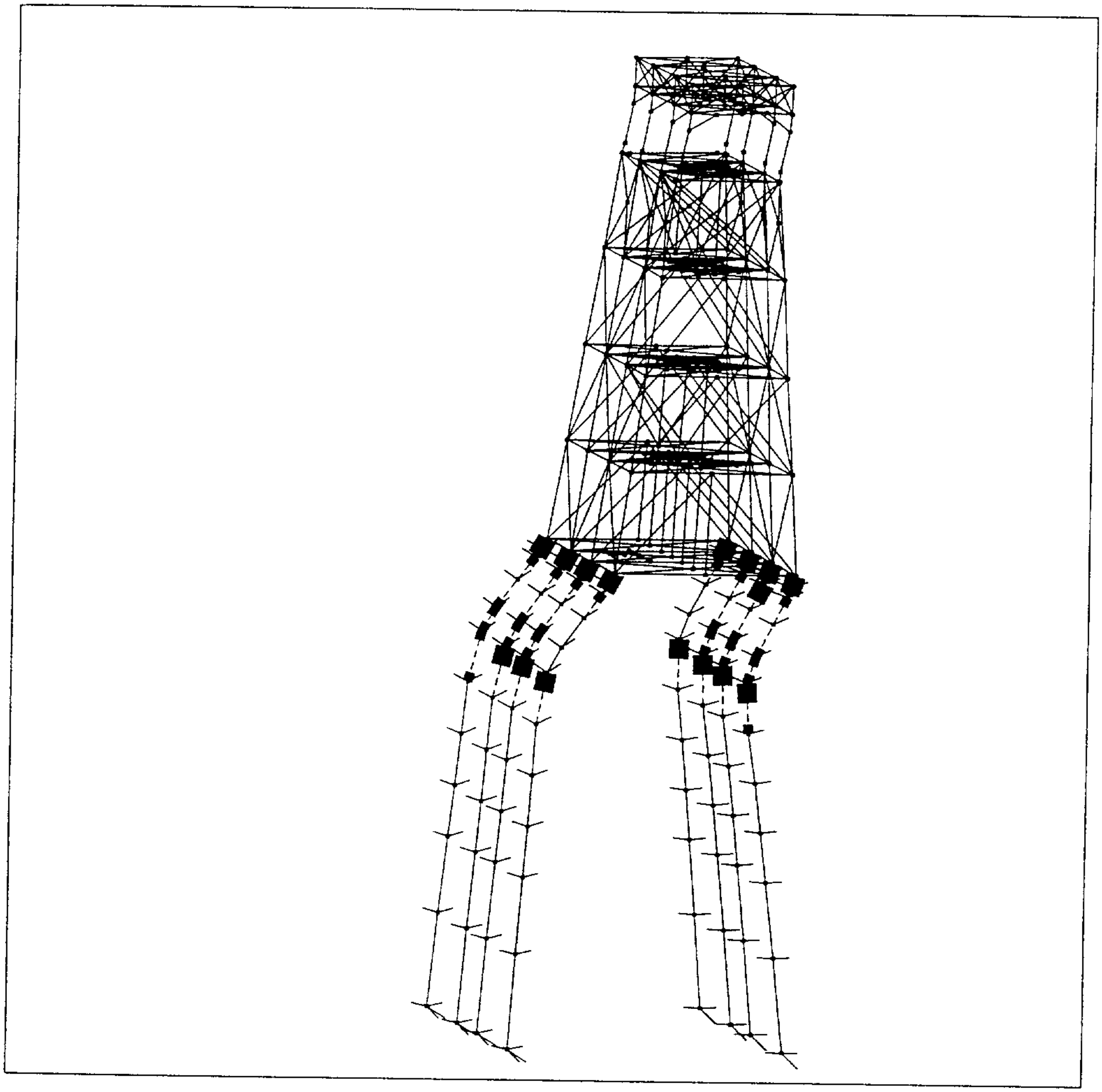


Figure 3-7: Pushover Analysis Results - Jacket Platform ST177B



CAP

Inelastic Events Legend

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

Figure 3-6: Pushover Analysis Failure Events Linear Jacket - Case 3 Analysis: Platform ST177B

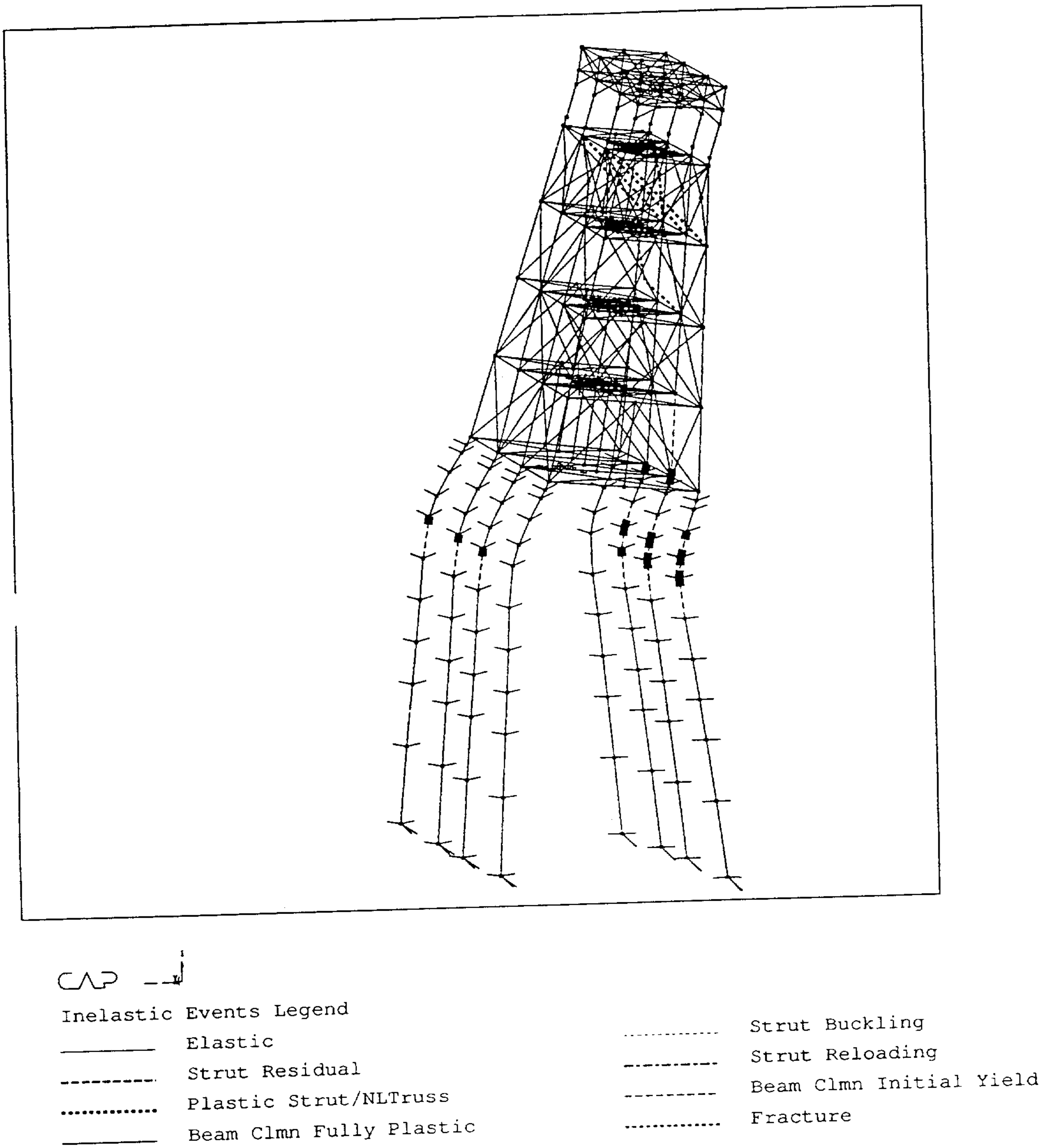


Figure 3-5: Pushover Analysis Failure Events - Jacket Platform ST177B

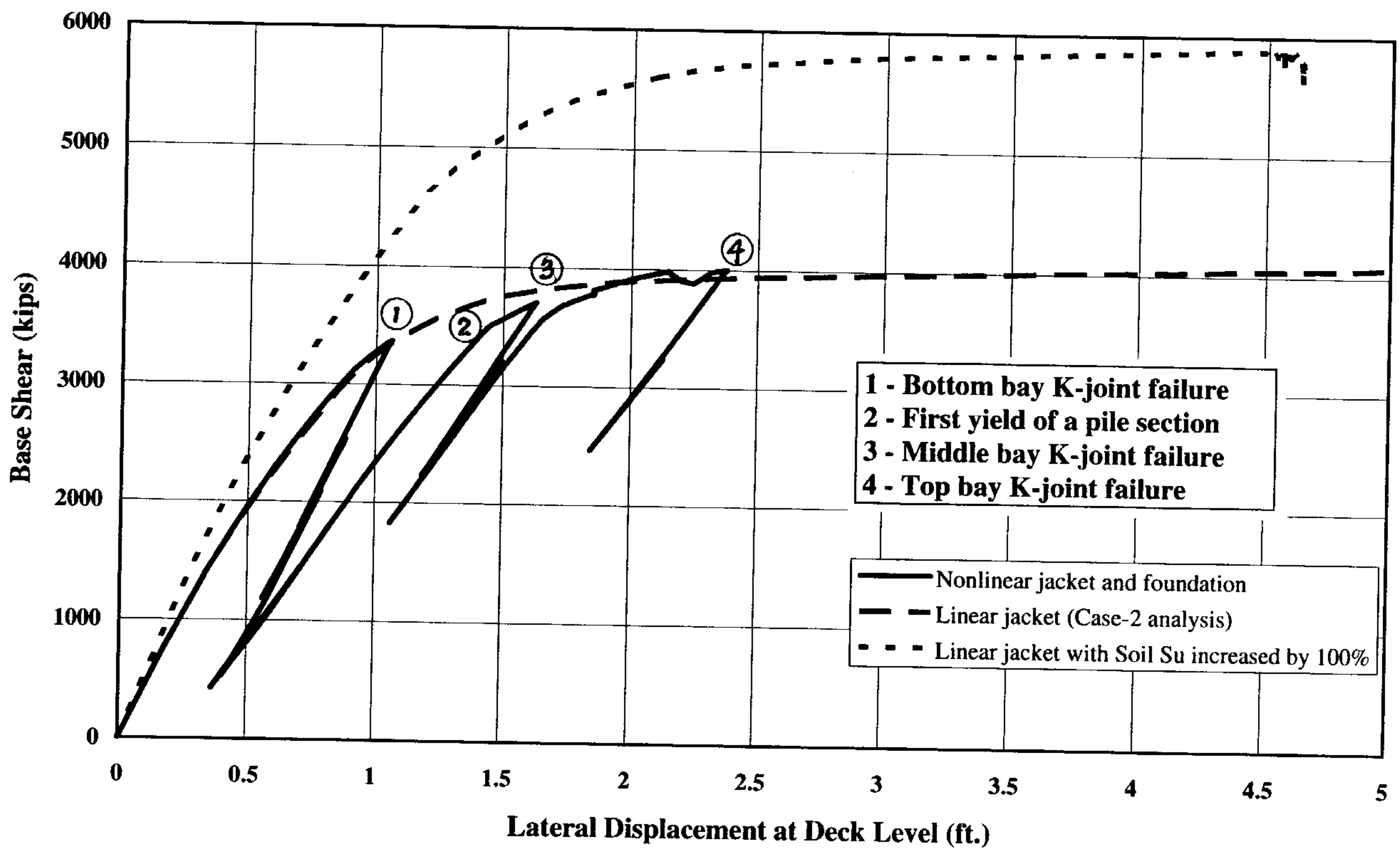
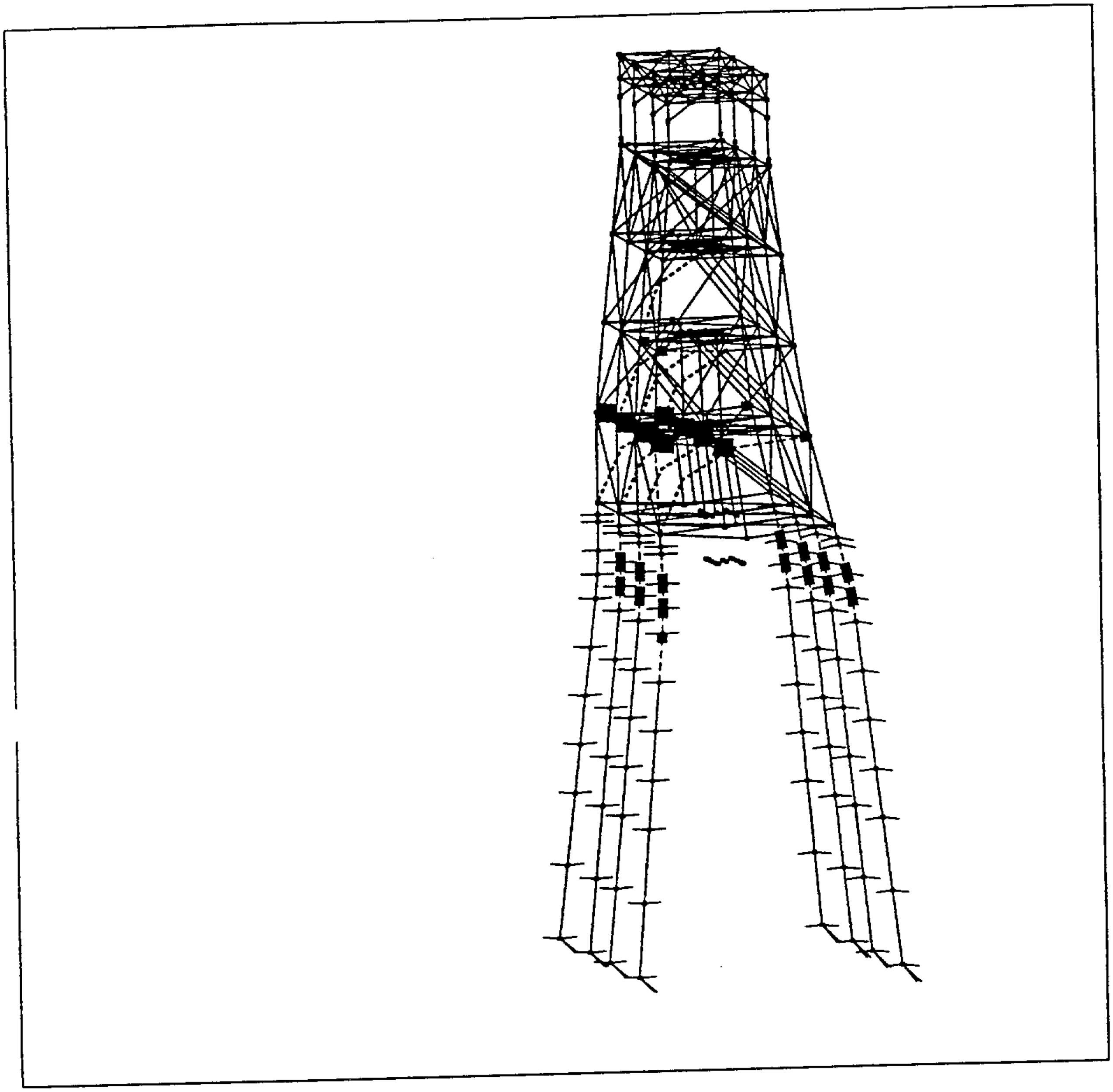


Figure 3-4: Pushover Analysis Results - Jacket Platform ST151K



CAP Load Step 288

Inelastic Events Legend

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

Figure 3-3: Pushover Analysis Failure Events - Jacket Platform ST151K

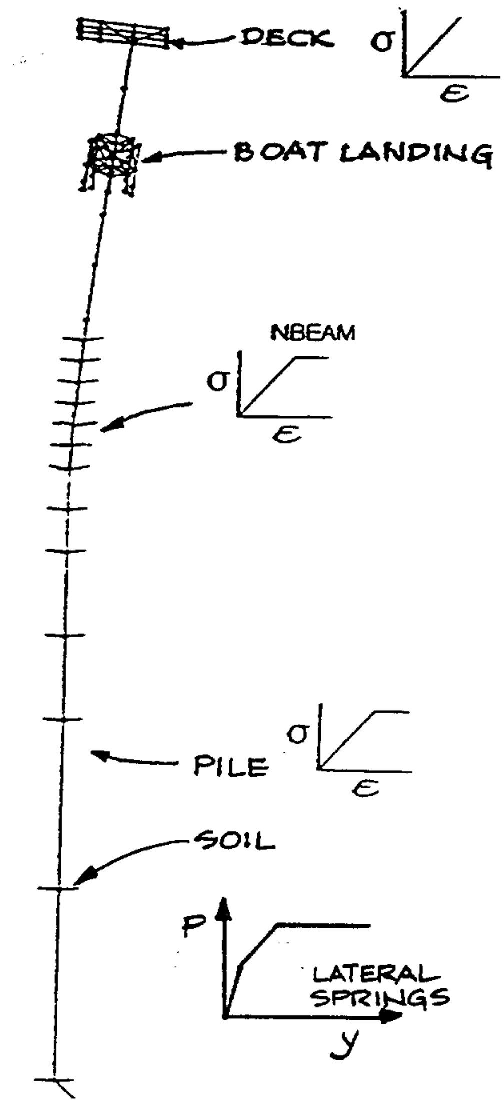


Figure 3-2: Nonlinear Analysis Computer Model - Caisson Platform

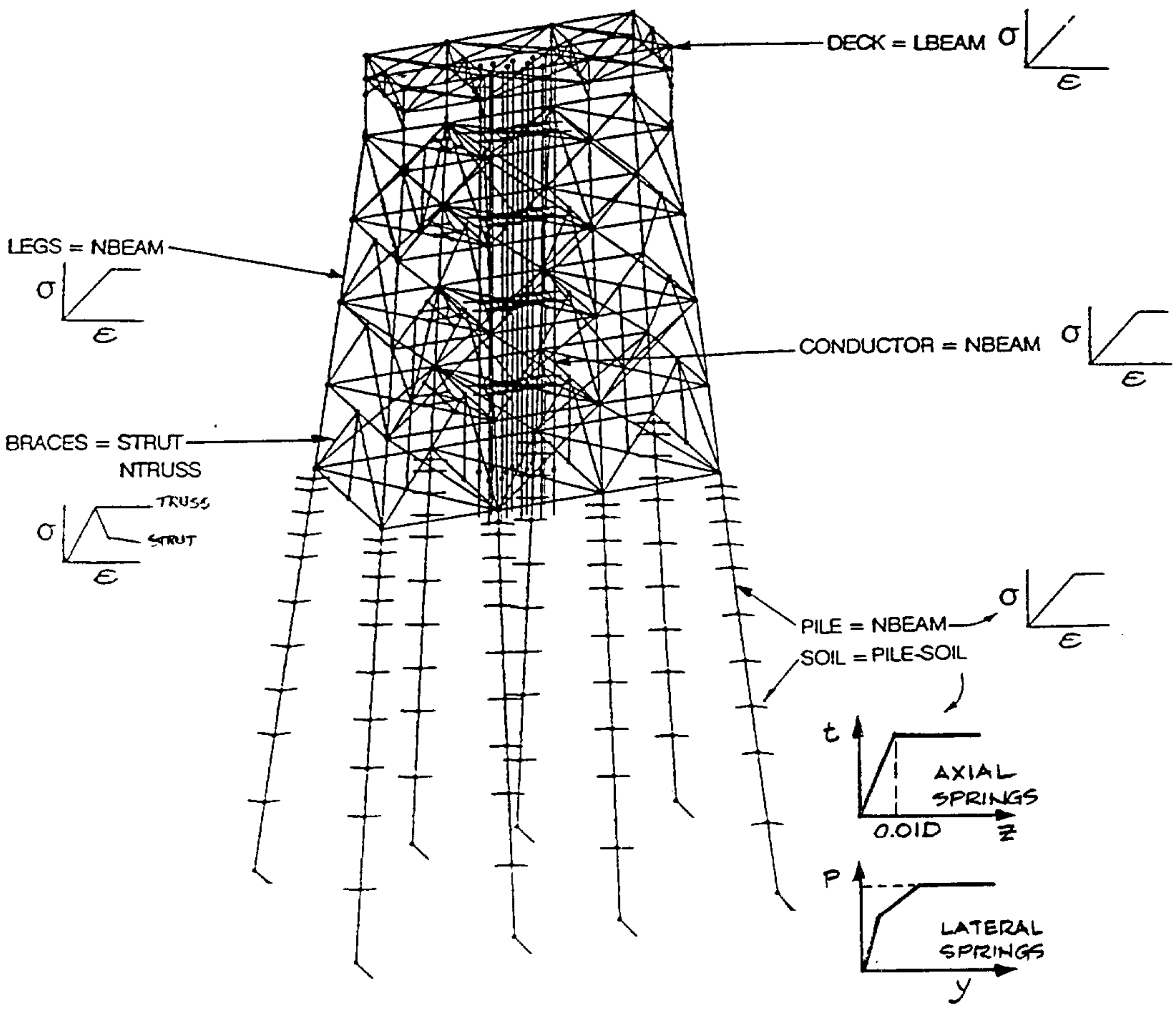


Figure 3-1: Nonlinear Analysis Computer Model - 8 Leg Steel Jacket Platform

likely shifting of yielding to lower thickness sections below (as obtained for SS136 — Table 3-2).

- Out of the four caisson platforms, based upon the results obtained, and considering non-availability of site specific soils, it was decided to include results for three caissons in the calibration work. Caissons SPelto 10, SS135, and SS136 (case with sand layer ignored) were selected.

Additional analysis done for the case in which the sand layer is ignored indicated the load levels at first yield, at the fully plastic section, and at ultimate capacity are 30 percent lower than the base case estimates.

3.5 SUMMARY

The capacity analysis results for three steel jacket platforms summarized in Table 3-1 indicate the following:

- In the case of both 8-leg jackets for the storm hour with the maximum wave heights in the diagonal direction, pullout and plunging of multiple piles are predicted at low load levels for the base case soil. Whereas, the field observations for both platforms indicated that no pile pullout may have occurred. In Section 2, various reasons were discussed which may increase estimates of pile/soil axial capacity. The actual capacity of these platforms may be higher than predicted.

- Both 8-leg platforms have similar jacket configurations and pile details. The lateral capacity of the pile/soil system is identical for both platforms and is estimated to be 25 to 60 percent higher than the axial capacity estimates of the foundation. In the case of ST177B, the soils are weaker than those of ST151K, but the lateral ultimate capacity estimates are same. These estimates of capacity (5,100 kips) are marginally higher than the predicted maximum Andrew base shear.

- The system factor for pile axial capacity is estimated between 1.2 to 1.3 for the 8-leg jackets, indicating that the load is redistributed from the overloaded to the underloaded piles. In the case of the 4-leg jacket, no load redistribution is noted (axial capacity system factor = 1.0). The system factor for the lateral capacity (plastification of pile sections) of pile/soil system is estimated to be 1.14 to 1.18, which indicates load differences and load redistribution among piles for both 4- and 8-leg jacket cases.

The capacity analysis results for four caisson platforms summarized in Table 3-2 indicate the following:

- The analysis results for caissons SS113 and SS136 tend to indicate that these two may be "unexpected damage" cases. However, the presence of a sand layer may be uncertain, because the soil information is not site specific. In case the sand layer does not exist, the load levels at yielding of sections and the ultimate capacity estimates will be lower due to change in the bending moment profiles and the

The expected maximum Andrew base shear is predicted as 65 kips, which is higher than the ultimate capacity estimate.

<u>Ship Shoal 113</u>: The soil parameters used in the original design were based on information from boreholes in adjacent blocks and caisson/conductor driving records in the vicinity. The design soil parameters considered no scour, and 10 ft. sand layer from 30 ft to 40 ft below the mudlevel. The wall thickness of caisson foundation varies from 1.75 inch at the mudline to 0.75 inch at 60 ft. below. The platform leaned by 5 degrees during Andrew.

Due to presence of the sand layer at 30 ft. below, the first yielding and full plastification are predicted in the 1.75 inch thick section at 20 ft. below the mudline at 186 kips and 214 kips respectively. Yielding of sections then extends to 1.5 inch thick sections few feet below the sand layer. The ultimate capacity is estimated as 235 kips, which is 70 percent higher than the predicted maximum base shear during Andrew.

<u>Ship Shoal 135</u>: The soil parameters used in the original design were based on borelog at the caisson site. The design soil parameters considered no scour, and all clay strata to depths exceeding 100 ft. below the mudline. The wall thickness of caisson foundation varies from 2 inches at the mudline to 1 inch at 40 ft. below. The platform leaned by 15 degree during Andrew.

The pushover analysis indicated fully plastic section occurs at 128 kips in 1 inch thick section at 40 ft. below the mudline, and the predicted ultimate capacity would be 148 kips, which is 12 percent higher than the predicted maximum Andrew load level.

The analyses for cases with increased shear strengths resulted in moderate increase in the ultimate capacity. The ultimate capacity increased by 12 and 20 percent for the cases with undrained shear strength (S_u) increased by 50 and 100 percent, respectively.

<u>Ship Shoal 136</u>: The soil parameters used in the original design were based on information from boreholes in four adjacent blocks. The design soil parameters considered 8 ft scour and a 10 ft. sand layer from 40 ft to 50 ft below the mudline. The wall thickness of the caisson varies from 2 inches at mudline to 1 inch at 45 ft. below. The platform leaned by 30 degree during Andrew.

The fully plastic section occurs at 197 kips and the ultimate capacity estimate is 224 kips, when the upper bound soil properties are used, which are higher than the predicted maximum Andrew load level of 126 kips. Due to the presence of sand layer at 40 ft. below, the first yielding and full plasticity occur in the 1.5 inch thick pile section just above the sand layer and extends to other sections above and below.

The predicted maximum base shear during Andrew is 4,460 kips, which is about 40 percent higher than the base case and Case 2 capacity estimates and about 13 percent lower than the Case 3 capacity estimate.

SS139 (T25): This is a 4-leg platform in shallower water depth (62 ft.), and was installed in 1969. The piles are of 36 inch diameter with 165 ft. penetration. The base case/nonlinear jacket model results in an ultimate capacity of 1,600 kips with the failure mechanism formed by events in the K-joints and first yield of sections in each pile (Figures 3-8 and 3-9).

The Case 2 analysis indicates that failure mechanism forms by full plasticity of sections at two levels of all piles, and none of the piles have an axial capacity failure event. This defines the lateral foundation capacity as 2,410 kips.

The Case 3 analysis, in which the formation of pile yielding/hinging is suppressed, indicates that plunging of two piles would occur at a load level of 2,500 kips, which defines the axial ultimate capacity for this 4-leg platform.

The predicted maximum Andrew base shear is 1,770 kips, which is lower than the Case 2 and Case 3 capacity estimates by 27 to 30 percent respectively.

3.4.2 Caisson Platforms

South Pelto 10: This is a 30 inch diameter caisson installed in 35 ft. water depth in 1985, with 180 ft. penetration below the seabed (the design penetration was 195 ft.). The platform was found damaged during Andrew, with a lean of 12 degrees. The breaking wave criteria governed the metocean parameters, with a wave height of 30.4 ft. and current speed of 6.7 ft/sec. The soil strata consist of soft to firm clay up to 166 ft. and tan fine sand beyond.

The load-displacement plot obtained for three cases: base case soil, soil shear strength increased by 50 percent and 100 percent are given in Figure 3-10. The ultimate capacity increases moderately with an increase in the undrained shear strength of soil.

The maximum bending moment occurs at 20 ft. below the seabed in 1.375 inch thick section, while the maximum bending stress occurs at 35 ft. below the seabed at the transition from 0.875 inch to 0.5 inch thick sections (Figure 3-11), resulting in first yield of this section at 38 kips lateral load. This section becomes fully plastic at a load level of 44 kips followed by the first yielding of another section 10 ft. above. At the ultimate load level of 48 kips, the lateral displacement at the deck level is 5.63 ft. and at the seabed is 1.2 ft. Beyond this load level, the tilting of the caisson will increase rapidly due to reduction in the moment carrying capacity of the pile section as local buckling occurs.

the mudline in 0.75 inch thick sections. The ultimate capacity is 25 percent higher than that estimated for the Case 2.

The predicted maximum base shear during Andrew is 4,860 kips, which is about 20 percent higher than the base case and Case 2 capacity estimates and is marginally lower than the Case 3 capacity estimate.

The pile head displacements at the onset of pile yield/hinge and pullout or plunging are of the order of a few inches. Therefore, due to limitations of their observation in the field inspections, it can not be said with conformity that the pile events may not have occurred during Andrew.

ST177B: This platform is similar to ST151K in structural configuration and was installed in 1965 in a water depth of 142 ft. It was severely damaged during Andrew, with damage reported in K- and X-joints. The joints failed by punching of chord, which was followed by its separation in the joint gap (2 inch) region for the K-joints. The X-joints failed by chord ovalization due to brace punching and in some cases longitudinal cracks formed in the chord.

Figure 3-5 shows the members with inelastic events and the deformed platform for the nonlinear jacket case (base case) close to respective ultimate capacity stages. For this model, the first event occurs by pullout of a pile followed by punching of X-joints in the jacket upper bay and first yielding of pile sections. The ultimate capacity stage is reached at 3,200 kips with events in multiple piles (Figure 3-7). At this stage, the lateral stiffness of the platform had become very low, thus defining the ultimate capacity.

The results obtained for the linear jacket (Case 2 analysis) indicate that the pullout of 3 piles and plunging of 3 piles would occur up to a load level of 2,970 kips, followed by first yield of pile sections at 3,200 kips. The ultimate capacity for this case is close to 3,200 kips. For this platform, the undrained shear strength of soil (Su) was lower than that for platform ST151K, which resulted in a lower axial capacity of the foundation system compared to that for the similar ST151K platform.

Case 3 analysis resulted in higher ultimate capacity of 5,100 kips defined by full plastification of several sections at two levels of all piles (Figure 3-6). The load levels at full plasticity of the first and multiple piles sections match that for ST151K (with undrained shear strength of soil higher than for ST177B), indicating negligible effect of variations in Su in the upper zone, which contributes to the foundation lateral capacity. The ultimate capacity estimate is higher by 60 percent than that for the Case 2.

The results summarized in Tables 3-1 and 3-2 are discussed for individual platforms in the following sub-sections.

3.4.1 Steel Jacket Platforms

<u>ST151K</u>: Figure 3-3 shows an overall view of platform ST151K, which is an 8 leg platform (external legs double battered) installed in 1963 in a water depth of 137 ft. The pile-leg annuli are grouted and the platform has K-joints in the broadside loading direction and diagonal braces in the end-on loading direction. The platform survived Andrew with no damage observed in the jacket and its foundation.

Figure 3-3 also shows members with inelastic events and the deformed platform shape (deflections have been amplified for better visual effect)close to its ultimate capacity stage for the base case analysis, for the diagonal direction pushover analysis. At this stage, several K-joints have inelastic events (followed by a fracture of the chord at the joint) and the first yield of a section occurs in seven piles. Figure 3-4 shows the force-displacement plot, with the displacements taken at the deck level. 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 pile pullout and K-joint failure occurs at an applied load of 3,350 kips. The platform then begins to respond in an increased nonlinear manner due to inelastic events in the jacket and redistribution of loads from the piles with the pullout events to other piles. Initial yield of a pile section occurs at an applied lateral load level of 3,470 kips and a fully plastic section does not occur. The platform ultimate capacity stage is reached at 4,000 kips with nonlinear events in several K- and KT-joints in the three jacket bays, pullout of two more piles and plunging of one pile.

The Case 2 analysis determined that pullout of the first pile is at 3,330 kips and multiple piles pullout/plunge up to a load level of 4,000 kips, just as they are for the Base Case. The first yield of a pile section occurs at 3,750 kips (higher than the corresponding load level for the Base Case) and no pile has a fully plastic section up to the ultimate load level of 4,070 kips. The load-displacement plot indicates that the linear jacket case envelopes the nonlinear jacket plots, thus indicating that the pile pullout events have a dominant role in limiting the ultimate capacity of the platform.

Figure 3-4 also shows that the ultimate capacity estimate increases by about 40 percent when the undrained shear strength of the soil is doubled and when the Case 2 model is used.

The Case 3 analysis, with suppressed pile pullout/plunging, results in the plastic section in the first pile at 4,340 kips and plastic sections in multiple piles at 5,000 kips, at 40 ft. below

- **Base Shear at Multiple Component Failure** — The global base shear at which multiple failure events occurred in each category of platform components (brace/joint, pile, soils, as applicable).

- **Base Shear at Ultimate Platform Capacity** — The global base shear at which the platform is considered as collapsed or to have reached maximum lateral load capacity with associated large deflections.

- **Collapse Modes** — The failure modes predicted from static pushover.

  Jacket Platforms — Frame failure indicated by failure events in a number of K-joints, braces, and/or first yield/hinge formation in the leg(s). Pile failures against lateral loads indicated by fully plastic hinge formation in multiple piles at one or two levels. Pile pullout/plunging failures indicated by pile axial loads exceeding the ultimate axial soil capacity. The ultimate capacity was considered to have been achieved when its stiffness was reduced to a low value and displacements at the deck level were 4 to 5 ft. The analysis was terminated at this stage.

  Caisson Platforms — Failure events are indicated by hinging of the caisson sections at some depth below the mudline. The collapse state is reached when local buckling is initiated in a caisson section.

- **Ratio of Ultimate Capacity (Ru) to Andrew Base Shear (BS)** — Provides an approximate estimate of the platform capacity compared to the Andrew base shear estimate.

- **System Factor** — Provides an estimate of the platform capacity beyond the first member failure, computed as the ratio of the ultimate capacity to the load level at the inelastic event in the first element (including pile first yield).

- **System Factor (Pile Lateral Capacity)** — Provides an estimate of the platform capacity beyond the first pile element with fully plastic section event, computed as the ratio of the load level at multiple pile plasticity events to the load level at the first pile with a plastic section.

- **System Factor (Pile Axial Capacity, Jacket Platforms only)** — Provides an estimate of the platform capacity beyond pullout or plunging of the first pile, computed as the ratio of the load level at multiple pile pullout/plunging events to the load level corresponding to the pullout/plunging of the first pile.

elements. The deck structure and boat landing framings were modeled as linear-elastic beam elements. The soil was represented by nonlinear p-y, t-z and torsional springs.

3.4 ANALYSIS RESULTS

The pushover analyses were done for only one storm approach direction. The wave direction for the storm hour with the maximum wave height was selected and the variations to this direction during other storm hours are likely to have minimum effect on platform failure modes and capacity estimates. During Andrew Phase I, some jacket platforms were examined for variation in capacity due to other wave directions, and minimum variations were noted.

There were no observable foundation damages to the jacket platforms, thus calibrating predicted failure events during storm hours with the maximum waves was considered appropriate.

The pushover analyses results for the three steel jacket platforms and for four caissons as necessary for calibration are summarized in Tables 3-1 and 3-2. The salient characteristics of the platforms were presented in Tables 1-1 and 1-2. The information found in columns of Tables 3-1 and 3-2 is as follows:

- **Background Information** — The first few columns describe the general characteristic of platforms.
- **Analysis Cases (Jackets)** — The analyses cases for jacket platforms: base case (nonlinear modeling for jacket, pile and soil elements), and Cases 2 and 3 (linear jacket model and nonlinear pile/soil elements).
- **Analysis Cases (Caissons)** — The analysis cases: base case soil, increased soil strength, or with thin sand layer ignored.
- **Expected Maximum Hindcast Base Shear** — An estimate (expected value) of the hindcast maximum global base shear during Andrew based on the maximum hourly hindcast wave height in the selected direction. The wave height and associated current are taken directly from the Oceanweather Andrew hindcast [Oceanweather 1992].
- **Base Shear at First Component Failure** — The global base shear at which the initial platform component (brace/joint, pile, soils, as applicable) failure event was predicted in the analysis. This information contains no factors of safety.

Section 4
Bayesian 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.

There are a number of parameters required to estimate structure capacity and environmental loads such as soil strength and density, and drag and inertia coefficients. In principle it is possible (with enough data) to "calibrate" the capacity and load models, that is determine correction factors on these nominal input parameters. Due to limited observed information (i.e., Hurricane Andrew observations) and a limited number of available platforms, it was not possible to calibrate many specific items of the analysis recipe/procedures and the calibration effort was limited to a global measure of the platform capacity. Thus, as in Andrew Phase I, the bias factor "B" is introduced as the correction to the computed "safety margin" of the platform, defined as the ratio of resistance (R) to load (S):

$$\left[\frac{R}{S}\right]_{true} = B\left[\frac{R}{S}\right]_{computed} \tag{4-1}$$

Thus, the "true" safety margin equals the "computed" safety margin (as per the assessment process) times a bias (or correction) factor, B. A value of B greater than 1.0 indicates (on average) that the current ultimate capacity analysis procedures provide conservative results. A value of B less than 1.0 indicates (on average) unconservative predicted ultimate capacity results. B is a random variable.

Andrew Phase I determined the posterior of B to have a mean of 1.2, which was the system bias factor applicable to the complete behavior of platform (jacket and foundation) and related to the analysis recipe followed in that project. The capacity analysis results corresponding to those presented for the Base Case (nonlinear jacket and foundation model) in Section 3 were used in Andrew Phase I. The primary differences between the analysis recipe used in the Andrew Phase I and this project were in the soil shear strength profiles used and the modeling of joints.

The capacity analysis results presented for the two 8-leg jacket platforms in Section 3 predicted failure and inelastic events in the pile/soil system at loads lower than the predicted maximum Andrew load level, and very small pilehead displacements at the ultimate capacity load level. For two damaged jackets investigated in this project, the damage in jacket frames were recorded but there was no conclusive observation of foundation behavior during Andrew suggesting that they did not fail or deform in a noticeable way. For this reason the capacity related to the jacket superstructure (with known behavior during Andrew) was separated from the foundation behavior.

The various factors which influence the lateral and axial behavior/capacities of the pile/soil system were discussed in Section 2. These and the earlier research findings discussed indicate that the biases associated with the foundation system vary significantly for the lateral and axial failure modes, thus these two modes shall also be separated. Thus the following were considered separately:

- Bias in the jacket superstructure, B_{ss}
- Bias in foundation lateral capacity, B_{fl}
- Bias in foundation axial capacity, B_{fa}

The base case capacity analysis presented in Section 3 provides results which include the effect of the foundation. The Case 2 analysis provides an estimate for the ultimate foundation capacity for lateral or axial directions individually whichever is dominant. The Case 3 analysis provides an estimate for the alternate mechanism by suppressing the Case 2 mechanism.

In this study, the foundation bias factors (lateral, B_{fl} and axial, B_{fa}) are to be determined. The capacity estimates presented in Table 3-1 for the base case are thus not used because the jacket mechanism was dominant. Instead, the estimates for Case 2 and Case 3 are used in calibration. The base case analysis results are useful however to estimate the behavior of the complete nonlinear model and identify any interaction effects of various failure modes.

Due to inconclusive observations regarding the foundation behavior during Andrew, three alternative interpretations for calibration are considered for both bias factors (B_{fl} and B_{fa}): In the first case, the observations are interpreted as non failure of a single pile. In the second case, the interpretation is non failure of multiple piles. The third case interpretation is failure of a single pile. Each of these possible interpretations is worked through to completion, i.e., determination of the mean and COV of the bias factors.

The Bayesian updating procedure to calibrate and evaluate the posterior distribution of B was developed in the Andrew JIP, and will be presented in a subsequent section as it applies to the foundation capacity calibration. The failure probabilities required to perform the Bayesian updating were evaluated using a rigorous probabilistic approach developed in the Andrew Phase I [PMB Engineering, 1993; Puskar et al, 1994] and is described in detail in Appendix B.

4.2 PRIOR DISTRIBUTION OF BIAS FACTOR, B

Due to the lack of observed foundation damage cases, the selection of an appropriate mean and COV for the "prior" distributions of B is more important than it was for the Andrew Phase I since it will have a greater effect on the outcome. The distribution of the "prior" will also serve to bound the bias distribution, and is based on earlier work of Dr. Wilson Tang [API PRAC 87-92, 1990; API PRAC 86-29B, 1988] which focused on uncertainties in the geotechnical parameters. Those were discussed in detail in Sections 2.2 and 2.3. Dr. Tang suggested to use normal distribution for the prior of B and recommended use of COV of 0.3 for both lateral and axial capacity:

		CLAY	SAND
Lateral Capacity:	Mean	1.0	0.8
	COV	0.2 - 0.3	0.3
Axial Capacity	Mean	1.3	1.0
	COV	0.3	0.5

4.3 CALIBRATION PROCEDURE DETAILS

An overview of the complete calibration methodology developed in the Andrew JIP and also followed in this project is presented in Figure 4-1. It consist of three stages:

- Capacity analysis
- Reliability analysis
- Bayesian updating

<u>Capacity Analysis</u>: Capacity analysis aims to establish the lateral load levels at different inelastic events in the jacket and its foundation and the ultimate capacity at platform collapse or at an excessive displacement stage.

The capacity analyses described in Section 3, were performed on three-dimensional models of the platforms using CAP [PMB Engineering, 1994]. The focus of the analysis was on establishing lateral loads (pushover) at the occurrence of various events in the jacket structure and foundation elements (pile sections, soil). In the case of jacket platforms, the inelastic/failure events in the jacket structure (e.g., brace/joint overstress, leg yielding) are also identified.

It is noted that the significant wave heights during storm hours reduce very sharply beyond the most intense 2 to 3 hours (which are within 20 degrees from the direction of the maximum storm hour). Thus the storm contribution before or after the most intense phase is unimportant since the analysis is not likely to predict any nonlinear events/failures due to other directions. The sensitivity analysis performed in Andrew Phase I indicated that the change in the approach angle did not change the load level significantly for the inelastic events, unless the failure modes shifted to frames in other orthogonal directions.

Therefore, for jacket platforms with no observed foundation damage, a capacity analysis for only one direction was considered adequate. For caisson platforms, because of their axial symmetric structural configuration, analysis for one direction was also sufficient.

At the first project meeting, the difficulties of observing foundation damage in post-hurricane inspections were discussed. The results from analysis of platforms presented at the meeting and reported in Section 3 indicated that the pilehead displacements at formation of fully plastic sections and at pullout of piles might be too small to be observable by a diver. The issue of which predicted inelastic events shall be calibrated was discussed. Previous work by Tang [1990] reported biases associated with full plasticity of a pile section (as in the API pile overload formulation) and pullout/plunging of a single pile.

The following failure modes are identified, which define the estimates of the lateral and axial capacities of the foundation of the jacket platforms:

- Lateral capacity: First yield of pile section
  Full plasticity of a pile section
  Full plasticity of several pile sections (system capacity)

- Axial capacity: Pullout/ plunging of a single pile
  Pullout/plunging of several piles (system capacity)

In the case of caisson platforms, the load levels at first yield of a section at fully plastic section, and at ultimate capacity were established, as given in Table 3-2. The caissons considered were all in the observed damage category.

<u>Reliability Analysis</u>: Using the Andrew hindcast information for several storm hours with higher wave heights and the applicable capacity analysis results, structural reliability analysis procedures were used to determine the probabilities of failure. The formulation developed by PMB during Andrew Phase I and described in Appendix B is followed. The process is automated by using the PF program developed by PMB.

The load (S) and capacity (R) in the computation of P_f are considered as random variables. The load, S represents the maximum load on the structure during Andrew. The load is represented by the base shear, *BS*, obtained for different combinations of wave height (h) and current (u) by an empirical formulation as given in the Appendix B. It requires platform specific base shear coefficients (C1, C2, and C3), applicable for a range of wave height and current for each direction considered. The best estimate of the capacity (R) is represented by the ultimate capacity of a platform obtained from the static pushover analysis.

A computer code called "C1C2C3" was developed during the Andrew JIP to define the base shear coefficients. The program performs a three-dimensional iteration and determines the best fit coefficients to the platform base shear computed for a set of different wave heights and corresponding current.

A computer code called "PF" was also developed during the Andrew JIP to compute the probability of failure for given values of load and capacity parameters. The formulation of "PF" includes a factor, b, which represents a different estimate of structural capacities (resistances) or different ratios of the best estimates of capacity to load. The probabilities of failure for each platform are evaluated for a range of values of b, e.g., 0.2 to 2.5.

The uncertainties and distributions of various quantities in equations B-1 and B-4 (Appendix B) are required for evaluating the probability of failure. The following distributions and variances were considered for this project:

Item	Distribution	Expected Value	COV
Capacity, R	Log-Normal	per analysis	0.20 for lateral pile capacity 0.30 for axial pile capacity
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 the 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 were considered sufficient in evaluating the probabilities of failure for a platform.

The values obtained from the "PF" program for different b values, represent the likelihood function given an observed failure during specific storm hours for a platform. The likelihood of the bias factor B less than b, given an observed failure of a platform due to a storm approaching from a particular direction is represented as follows:

$$\begin{aligned} \text{lk}\,(b \mid \text{failure}) &= P[\text{failure} \mid b] \\ &= P_f\,(b) \end{aligned} \tag{4-2}$$

in which $P_f(b)$ is the probability of failure of a platform at $B = b$.

Bayesian Updating: The objective of calibration is to modify the prior distribution on "B" in a manner consistent with the behavior observed during Andrew. The updating is based on the Bayes theorem of probability theory [Benjamin and Cornell, 1970; Moses, 1976; Tang, 1981] which states:

$$f''_B\,(b) \;\propto\; f'_B\,(b)\ \text{lk}\,(b \mid \text{new information}) \tag{4-3}$$

in which f'_B (b) is the "prior" distribution of bias factor, B; f''_B (b) is its "posterior" distribution, and lk(b|new information) is the "likelihood function" on b which reflects the information about b contained in the new observation. The likelihood function depends upon the observed state of a platform, i.e., survived, damaged, or failed during Andrew.

For a failure case, the likelihood function is given by equation 4-2. For a survival platform (no observed damage) case, the likelihood function becomes:

$$\begin{aligned} \text{lk}\,(b \mid \text{survival}) &= P[\text{survival} \mid b] \\ &= 1 - P_f(b) \end{aligned} \tag{4-4}$$

For a damaged platform case, the likelihood function is the probability that the observed damage lies in the same fractional interval of the capacity to load ratio as predicted by the pushover analysis. The predicted ratios corresponding to the observed damage and one step more damage (i.e., one more failed component) are denoted α_1 and α_2 respectively. The resulting likelihood function for a damage platform case would be:

$$\text{lk}\,(b \mid \text{damage}) = P_f\,(\alpha_1\, b) - P_f\,(\alpha_2\, b) \tag{4-5}$$

The above likelihood functions, for a range of values of *"b"*, represent the information about *B* contained in the observed behavior of an individual platform. The combined likelihood function of *B* given the observed behavior of a number of platforms with a combination of survivals, damages, and failures is obtained by direct multiplication of the likelihood curves for each of the individual platforms as follows:

$$lk(b \mid n\ observations) = \prod_{\substack{platform \\ i}}^{n} [lk(b \mid observations)_i] \qquad (4\text{-}6)$$

The combined likelihood function developed for a number of platforms in the last step is then used to establish distribution of bias factor, B, using relationship 4-3. The mean and COV of the posterior distribution are then determined. The change in the mean value of prior of B identifies bias (conservatism or non-conservatism) in the load and resistance recipe.

In this project, the attempt is to establish distribution of two bias factors associated with two foundation failure modes: pile shear and axial pullout/plunging. The uncoupled foundation capacity estimates are used. Correlation between seastates, load level, and capacities in different directions have been neglected.

4.4 CALIBRATION RESULTS -APPLICATION TO PLATFORMS

The calibration approach described in Section 4.3 was applied to three steel jacket platforms and three caissons identified in Section 3. The capacity analysis results for these jackets and caissons were summarized in Tables 3-1 and 3-2.

Additional wave runs were performed on models of these platforms for use in determination of the coefficients to define base shears for use in the probability analysis. Representative wave and current combinations were run past the platform computer models and resulting base shears were determined. Twenty to thirty wave runs were required for each platform. The C1, C2, C3 coefficients used in the base shear formulation (Appendix B) were then determined using the C1C2C3 program.

The likelihood functions given an observed failure of platforms were then determined for each platform. The analysis was performed using two foundation ultimate capacity (R_{uf}) estimates given in Tables 3-1 and 3-2, the hindcast seastate data for 4 to 5 storms hours with maximum significant wave heights (H_s), the uncertainties in parameters given in Section 4.3.

The factor, b, within the formulation of probability of failure (Appendix B) was varied from 0.2 to 2.5 and the PF program was used to develop the probabilities of failure at each b value. The program determines the optimized integration limits and points for various parameters for a given value of b. The analyses was done using the optimized integration limits for different ranges of b values to determine probabilities of failure (P_f). A plot was generated of the resulting values of P_f for different b values, known as the "likelihood function" given an observed failure of a platform. The likelihood functions obtained for all jacket platforms and caissons from the PF are given in Fig. 4-2(a) for case with foundation ultimate lateral capacity (R_{ufl}) and COV as 0.2 and in Fig. 4-2(b) for case with foundation ultimate axial capacity (R_{ufa}) and COV as 0.3.

These likelihood functions were then used to perform Bayesian updating utilizing the predicted capacity analysis results and observed behavior information for platforms. The distributions of prior of B's were assumed as a mean of 1.0 and COV of 0.3 for the lateral capacity estimate and as a mean of 1.3 and COV of 0.3 for the axial capacity estimates. A sensitivity study performed in Andrew Phase I indicated that the posterior distribution of B is relatively insensitive to the COV of the prior.

The results obtained from Bayesian updating for lateral and axial capacities of platforms are presented below:

4.4.1 Bias Factor – Lateral Pile Capacity

The posterior distributions of bias factor (B_{fl}) for lateral pile capacity were developed separately for both steel jacket platforms and caissons and combined later to determine the limiting effect of caissons. The analysis results indicate that foundation failures (lateral and axial) can occur at relatively small (and perhaps unobservable) displacements. Therefore, an observation of an "unfailed" foundation have several possible interpretations. Three calibration cases were identified for Bayesian updating. In the case of caissons (failure cases), only one case with development of a fully plastic section was considered in calibration.

The calibration results obtained are summarized in Tables 4-1 to 4-3 for jacket platforms and caissons individually, and their combined effect.

<u>Jacket Platforms</u>

The calibration is done for the following possible interpretations of the observed foundation behavior:

Case A: Fully plastic section event in a single pile did not occur

Case B: Fully plastic section events in several piles did not occur

Case C: Fully plastic section event in one pile did occur

Table 4-1 presents the load levels at nonlinear events in pile/soil and foundation ultimate capacity taken from Table 3-1 for the above three calibration cases. The likelihood functions in Fig. 4-2(a) were then developed for three cases. Figures 4-3, 4-4, 4-5 provides the applicable likelihood functions for the three cases.

Figures 4-6(a) to 4-6(c) provided the prior and posterior distributions for B_{fl} for the three cases for jacket platforms. It is noted that the shift in the mean of prior of B_{fl} is similar for both ST151K and ST177B. For ST177B, B_{fl} shifts to 1.20 and 1.17 for Cases A and B, respectively. The shift due to platform SS139(T25) is less than for the other two platforms. The COV's of posterior of B_{fl} due to individual platforms are about 0.21 for all cases.

The posterior distribution of B_{fl} has a mean of 1.32 for Case A when cumulative effect of all three platforms is accounted. For Case B when multiple events are also not observed, the mean of B_{fl} shifts to 1.26. For Case C, with the assumption that only first pile plasticity event is observed, the mean of B_{fl} does not shift. The COV for posterior of Bfl averages to approximately 0.17 for all cases.

Caissons

Table 4-2 presents the load levels at nonlinear events in caisson foundation taken from Table 3-2 for the calibration case with observing fully plastic section event. The likelihood functions (observed failure case) in Figure 4-2(a) are applicable with some shift due to change in the load level calibrated. Figures 4-3 to 4-5 include the applicable likelihood functions for the caissons.

Figure 4-7 provides the prior and posterior distributions for B_{fl} for the three caissons. It is determined that South Pelto 10 caisson does not shift the prior of B_{fl}. Caisson SS135 shifts the mean of posterior of B_{fl} to 0.88 and caisson SS136 shifts B_{fl} to 0.83. The shift in B_{fl} due to all three caissons is to 0.77 with a COV of 0.29.

Jackets and Caissons Combined

Figures 4-3 to 4-5 also provide the combined likelihood functions for three calibration cases (identified under jacket platforms) including effect of all jackets and caissons. The caisson likelihood functions are the same for all three calibration cases.

Figures 4-8(a) to 4-8(c) provides priors and posteriors of B_{fl} and Table 4-3 summarizes results for platforms under two groups and their combined effect on the posterior of B_{fl}. The mean of posterior of B_{fl} for jackets (Case A) reduces from 1.32 to 1.09 due to including caissons effect. Whereas, Case B mean posterior of B_{fl} for jacket case reduces from 1.26 to 1.04 due to caissons effect. For Case C, the reduction in mean of posterior of B_{fl} (jackets case) is from 1.0 to 0.91 due to caissons.

The third set of results present effect of considering caissons on the first set of the bias factors based on the steel jacket platforms alone. These results shall be considered with caution due to differences in the characteristics and behavior of jackets and caissons. Some of the reasons are discussed as below:

- Variations in the pilehead fixity for the jacket and the caisson at the mudlevel due to the characteristics and behavior of the superstructure.
- Variation in the failure modes: The ultimate lateral failure mode for the jacket platforms in many cases would be formation of fully plastic sections (hinge) in multiple piles at two levels — near mudline and some depth below. Whereas, in case of caissons the failure mode is the formation of fully plastic sections at some depth below the mudline.
- Variation due to platform loads: The bias factor represents the correction factor to the ratio of the predicted platform capacity to loads. The load computation for the jacket platforms involve a number of other factors (such as conductor shielding factor, current blockage factor, and others) and have complex physical characteristics compared to the caissons. Therefore the contributions of loads in the two systems (jackets and caissons) in the predicted ratios of ultimate capacity and loading effects (R_u/S) would vary for the two systems.

The scope of this study did not include identifying the sources of such differences and quantifying their effects on the resulting bias factors. The above reasons indicate that an effort to investigate the differences in the characteristics and behavior of the jacket platforms and caissons and variations in their contributions on the bias factors may be useful in future studies.

The calibration results presented for the third set shall be considered as a sensitivity study only. These results are intended to demonstrate the likely trend of shift in the bias factor based on jackets. The caissons were considered due to similarities in their lateral behaviors with the jacket platforms and due to non-availability of any jacket platforms with observable foundation damages to the project. But it was not intended to develop bias factor applicable to caissons and more specifically these bias factors demonstrate the likely trend for the jacket bias factor, when jacket platforms with foundation damage are included (if considered in later investigations).

In case of caissons, the second set of bias factor is more appropriate. Though this shall also be considered with caution due to being based on only three caissons with all under the observable damages category. In case some caissons with observed survivals (unexpected or likely) were available, this bias factor would likely increase.

4.4.2 Bias Factor (B_{fa}) – Axial Pile Capacity

The posterior distribution of bias factor (B_{fa}) for axial pile capacity was developed for steel jacket platforms only. The analysis results indicate relatively small (and perhaps unobservable) displacements at axial failure of foundation. Therefore, an observation of an "unfailed" foundation has several possible interpretations. The following three cases were identified for calibration of the axial capacity:

Case D:	Pullout or plunging event for first pile did not occur
Case E:	Pullout/plunging events in several piles did not occur
Case F:	Pullout/plunging event in one pile did occur

The calibration results obtained are summarized in Table 4-4. Table 4-4 presents the load levels at pullout/plunging events taken from Table 3-1 for the above three cases. The likelihood functions in Fig. 4-2(a) were then developed for each calibration case. Figures 4-9(a) to 4-9(c) provide the applicable likelihood functions.

Figures 4-10(a) to 4-10(c) provide the prior and posterior distributions for B_{fa} for the three cases. It is noted that the shift in the prior of B_{fa} (mean = 1.3) is mostly due to the two 8-leg platforms and the mean of posterior of B_{fa} are 1.52 and 1.63 for Case D and 1.49 and 1.57 for Case E. Due to SS135, a 4-leg platform, the posterior of B_{fa} shifts to only 1.37. Whereas, for Case F the most shift in mean of B_{fa} (to 1.53) is due to ST177B.

The posterior distribution of B_{fa} has a mean of 1.73 for Case D (not observing pullout event for the first pile) when cumulative effect of all three platforms is included. For Case E when pullout/plunging events are not observed in several piles, the shift in mean of B is from 1.3 to 1.66. For Case F, with the assumption that pullout is observed in one pile and subsequent predicted events were not observed, the mean of B_{fa} shifts to 1.53 from the prior mean of 1.3. The COV for posterior of B_{fa} reduces to approximately 0.18 for all cases.

4.5 SENSITIVITY OF PRIOR DISTRIBUTIONS OF BIAS FACTORS ON THEIR POSTERIORS

In this study, the prior distributions of B_{fl} (for lateral pile capacity) was assumed as a mean of 1.0 and COV of 0.3. The prior of B_{fa} (for axial pile capacity) was assumed as a mean of 1.3 and COV of 0.3. A sensitivity study performed in the Andrew Phase I, with 13 steel jacket platforms investigated indicated that the posterior distribution of B is relatively insensitive to the COV of the prior.

The sensitivity of change in the mean and COV of priors of B_{fl} and B_{fa} on their posterior distributions are investigated. The results obtained are summarized below:

Case 1: Variation in COV of Prior of B_{fl}:

Figure 4-11(a) shows the effect of variation in the prior COV from 0.3 to 0.2 and 0.4 for the Case A calibration. The resulting posteriors including the effect of three jackets have a mean of 1.19 and 1.44 for prior COV of 0.2 and 0.4 respectively, compared to the mean of 1.32 for the prior COV of 0.3.

Figure 4-11(b) shows the effect of variation in COV of prior for the Case B calibration. The resulting posteriors with the effect of three jackets have a mean of 1.15 and 1.38 for prior COV of 0.2 and 0.4 respectively, compared to the mean of 1.26 for the prior COV of 0.3.

These figures also include the results for the posterior means for the combined jackets and caissons case.

Case 2: Variation in the Mean of Prior of B_{fl}:

Figure 4-12 shows the effect of variation in the mean of prior from 1.0 to 0.8 and 1.2 for the Case A calibration. The resulting posterior distributions with the effect of three jackets have a mean of 1.13 and 1.50 for the prior mean of 0.8 and 1.2 respectively, compared to the posterior mean of 1.32 when the prior mean is 1.0. This figure also includes the results for the combined jacket and caisson cases.

Case 3: Variation in COV of Prior of B_{fa}:

Figure 4-13(a) shows the effect of variation in the prior COV from 0.3 to 0.2 and 0.4 for calibration cases D, E, and F. The resulting posterior distributions for the Case D, with the combined effect of all three jackets, have a mean of 1.55 and 1.87 for the prior COV of 0.2 and 0.4 respectively, compared to the mean of 1.73 for the prior COV of 0.3.

Case 4: Variation in the Mean of Prior of B_{fa}:

Figure 4-13(b) shows the effect of variation in the mean of prior from 1.3 to 1.1 and 1.5 for calibration cases D, E, and F. The resulting posterior distributions for the Case D, with the combined effect of all three jackets have a mean of 1.53 and 1.90 for the mean of prior of 1.1 and 1.5 respectively, compared to the mean of 1.73 for the mean of the prior as 1.3.

Table 4-5(a) summarize the ranges of the mean values and COV's of the posterior of B_{fl} for Case A and Case B calibrations for the corresponding mean values and/or COV's of the prior presented in Figures 4-12 (a) and 4-12(b). For the 3-jacket only case the mean of the posterior will vary from 1.13 to 1.50 and the COV between 0.14 and 0.21.

Table 4-5(b) summarize the ranges of the mean values and COV's of the posterior of B_{fa} for calibration cases D, E, and F for the corresponding mean values and COV's of the prior presented in Figure 4-13(a) and 4-13(b). For Case D the mean of the posterior will vary from 1.53 to 1.89 and the COV between 0.14 and 0.17. For Case E, the posterior mean will range from 1.46 to 1.82 and the COV from 0.15 to 0.19.

The above results indicate a relatively greater effect of variations in the prior mean and COV on the posteriors of both B_{fl} and B_{fa} than that determined in the Andrew Phase I project. This may be due to small sample of platforms investigated in this project and all three jackets being under survival category and all three caissons under damage/failure category.

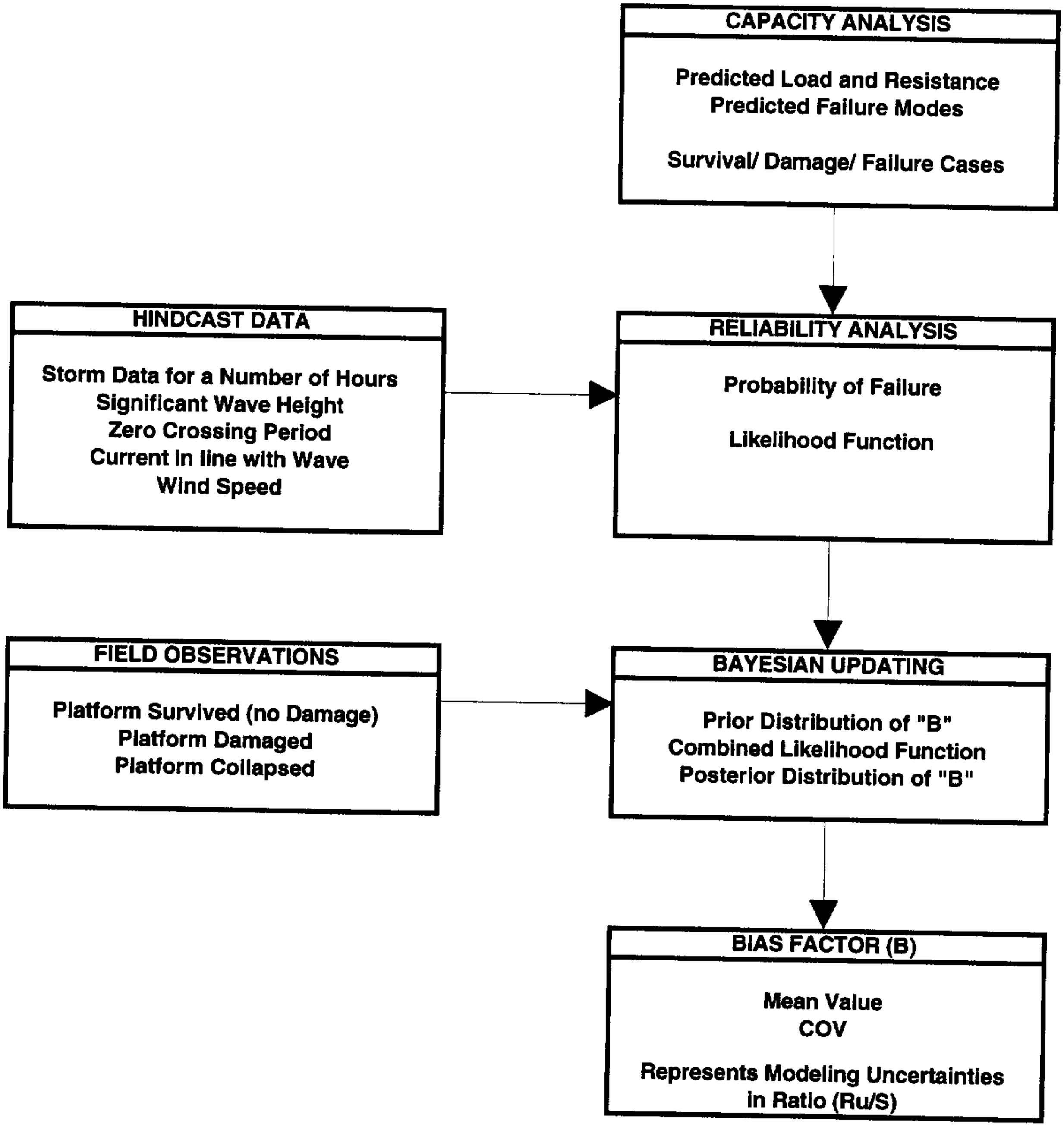


Figure 4-1: Calibration Methodology

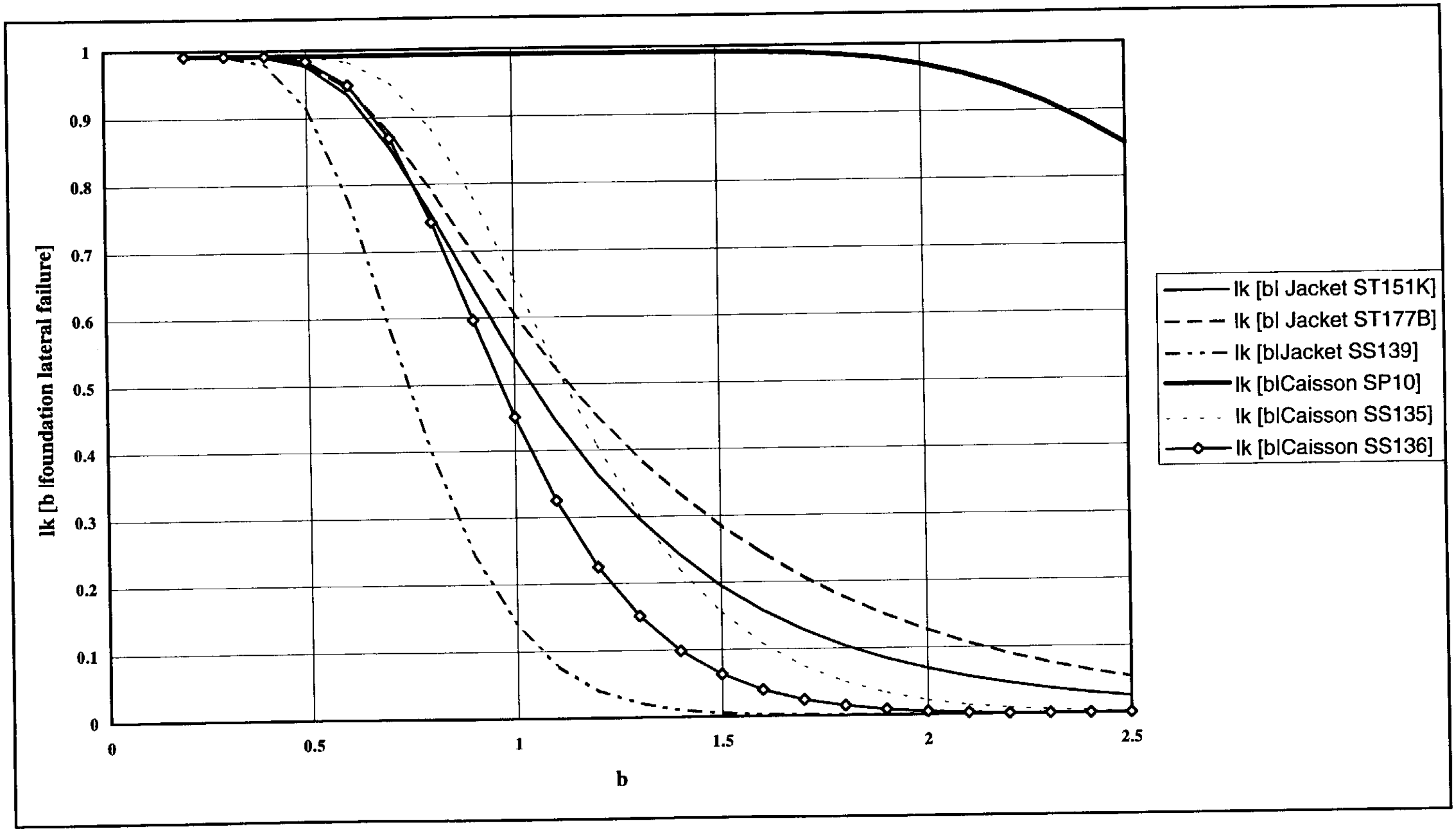


Figure 4-2(a): Likelihood Functions at Foundation Ultimate Lateral Capacity (R_{ufl})
- Given Foundation Failure of a Platform

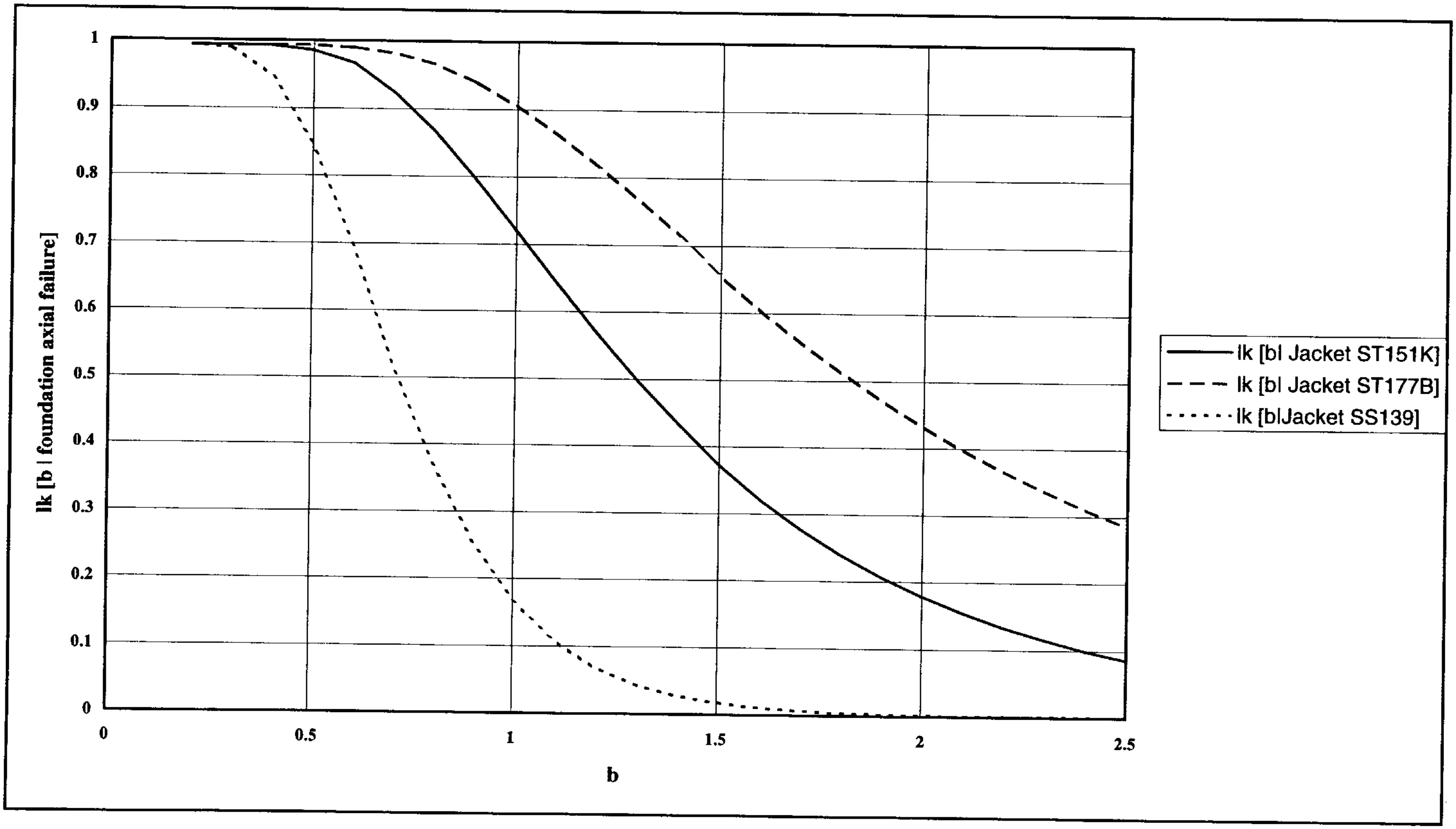


Figure 4-2(b): Likelihood Functions at Foundation Ultimate Axial Capacity (R_{ufa})
- Given Foundation Failure of a Platform

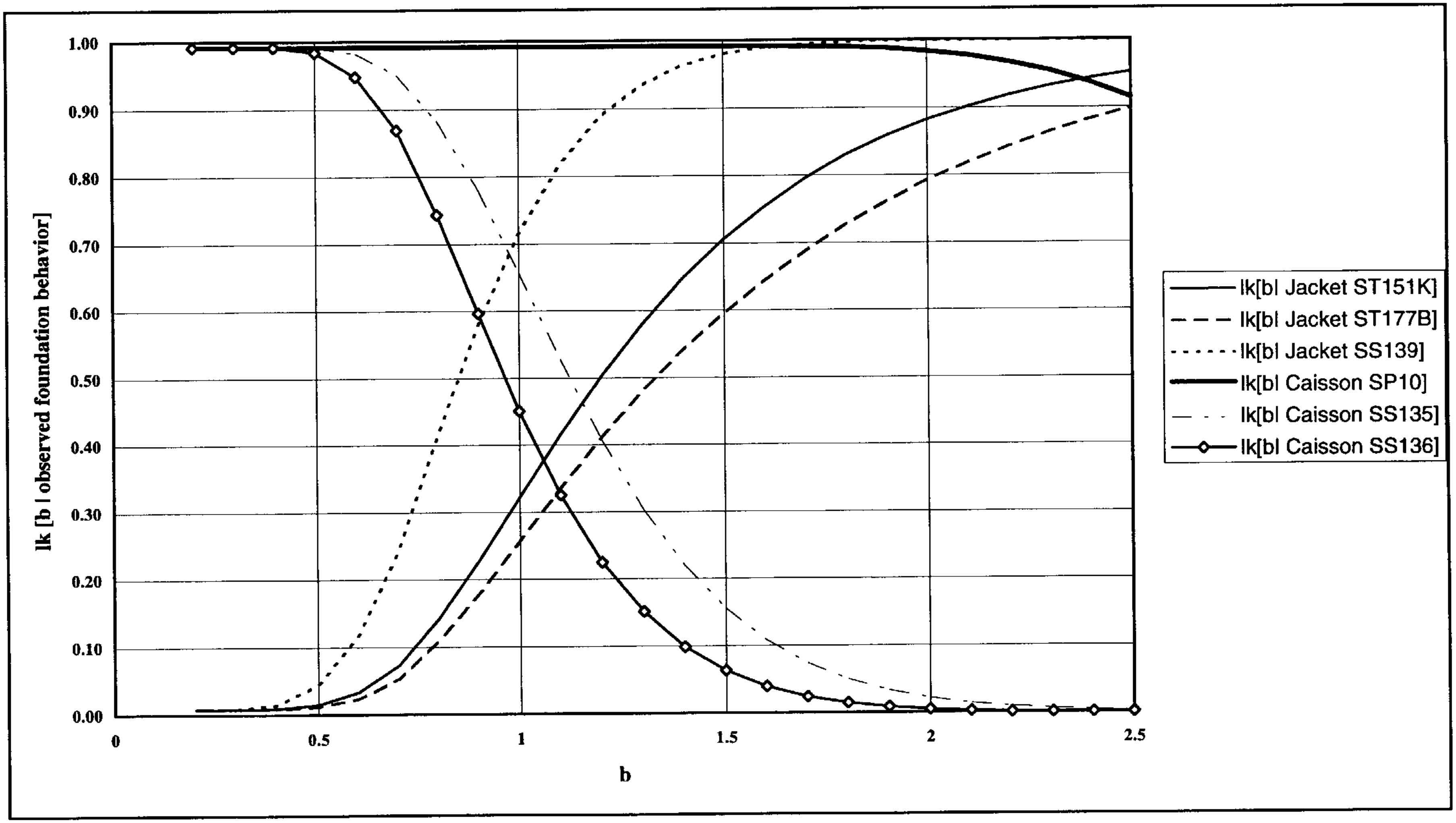


Figure 4-3(a): Individual Likelihood Functions - Foundation Lateral Capacity Case A

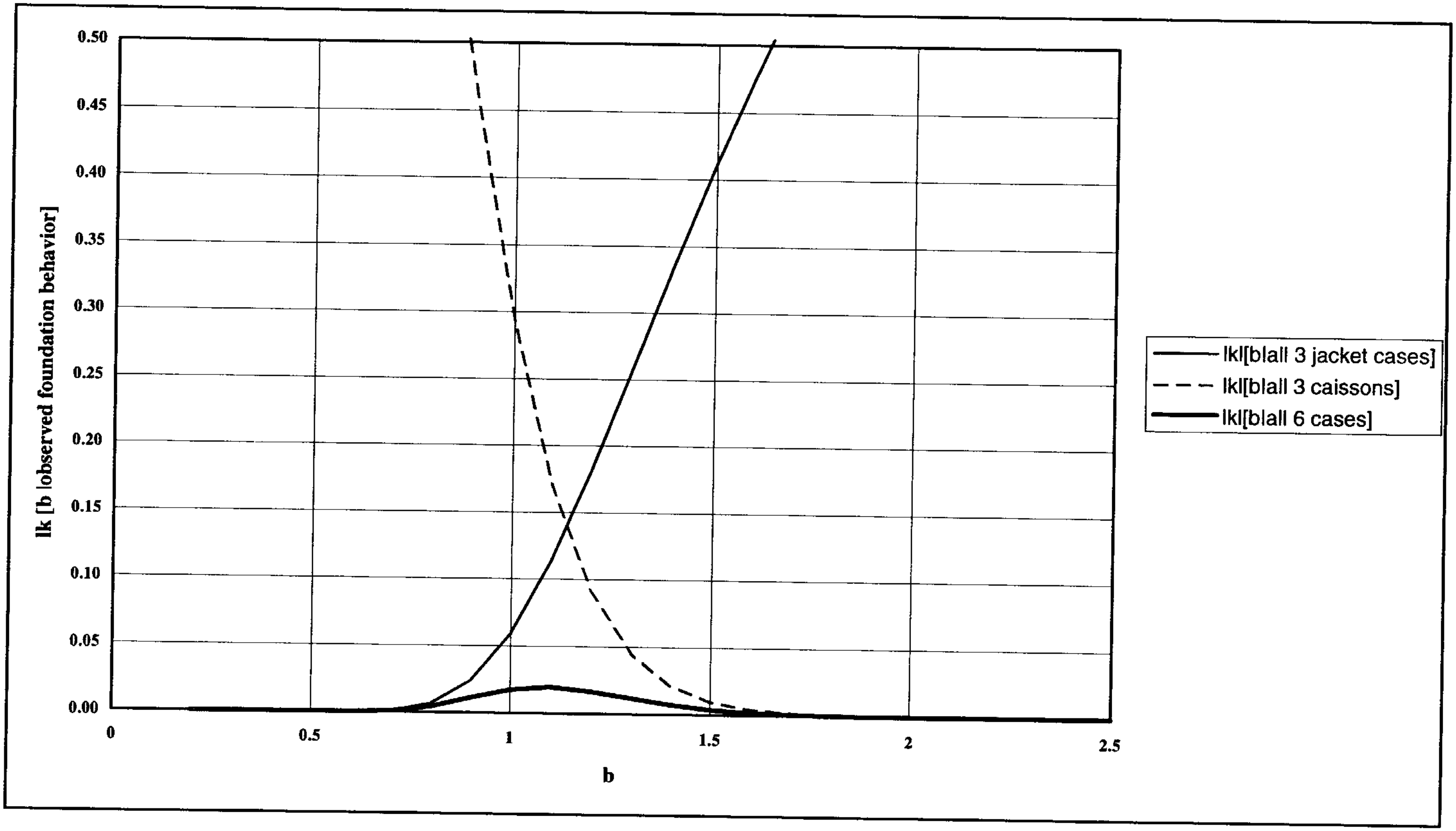


Figure 4-3(b): Group Likelihood Functions - Foundation Lateral Capacity Case A

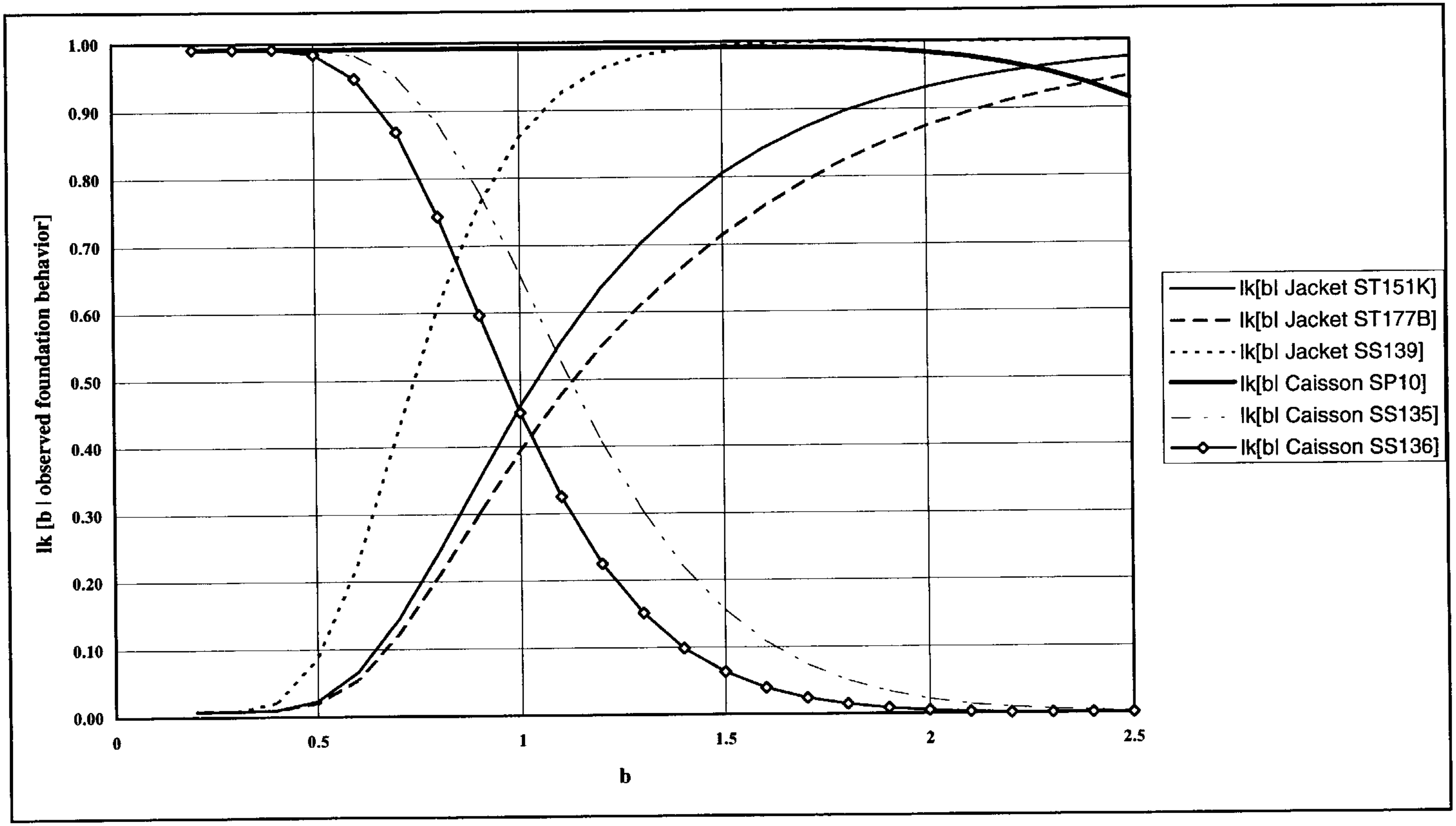


Figure 4-4(a): Individual Likelihood Functions - Foundation Lateral Capacity Case B

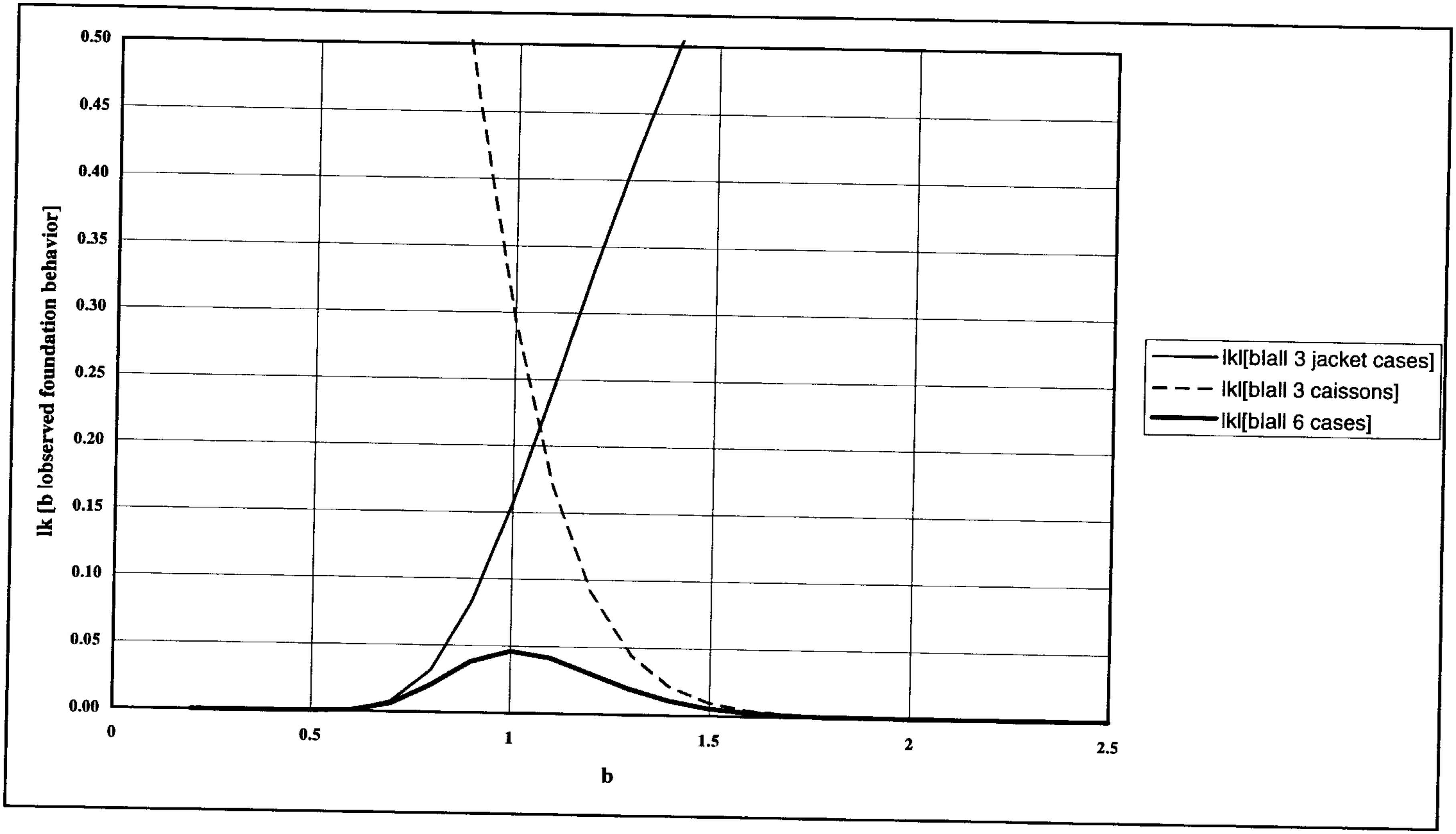


Figure 4-4(b): Group Likelihood Functions - Foundation Lateral Capacity Case B

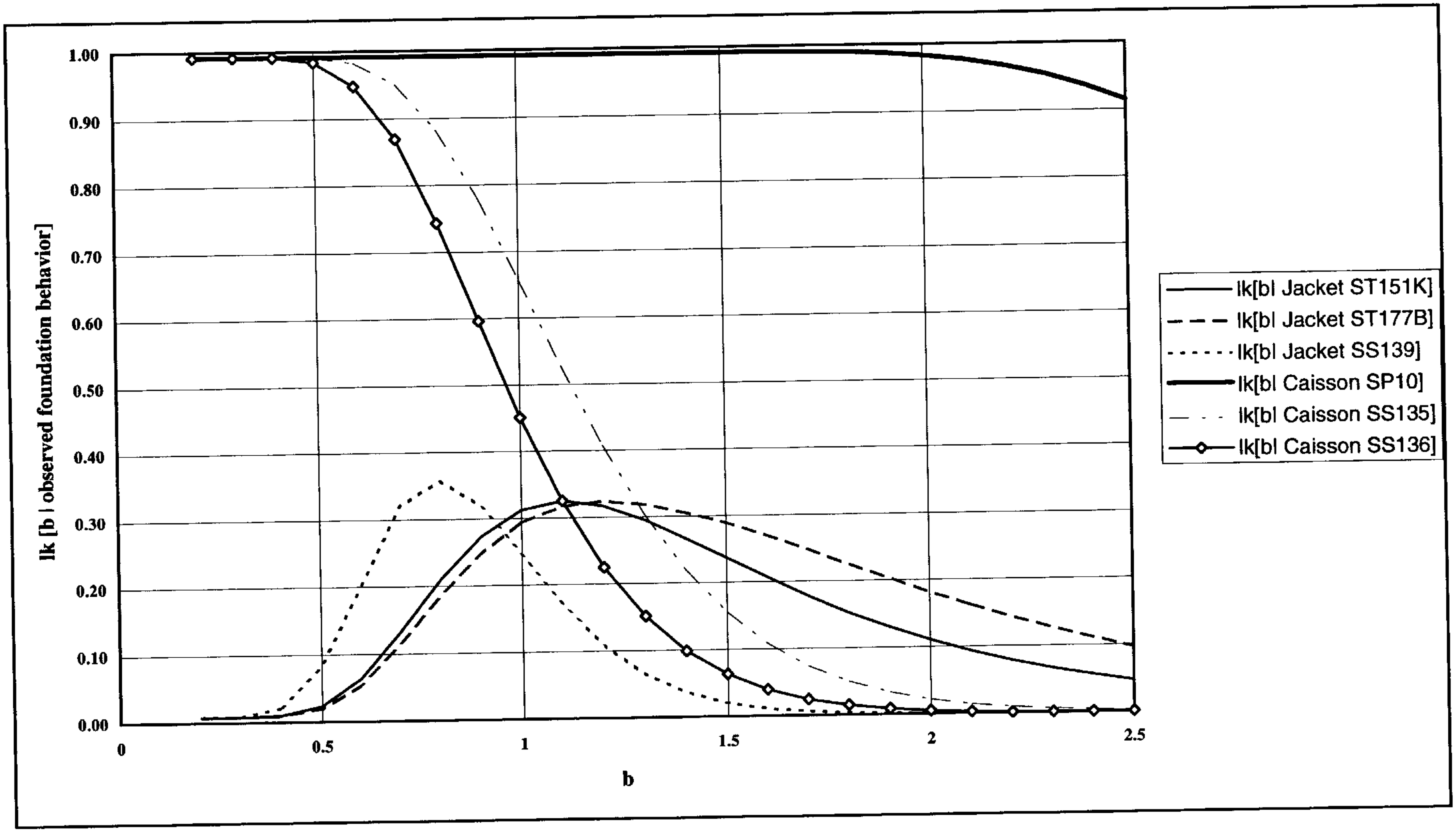


Figure 4-5(a): Individual Likelihood Functions - Foundation Lateral Capacity Case C

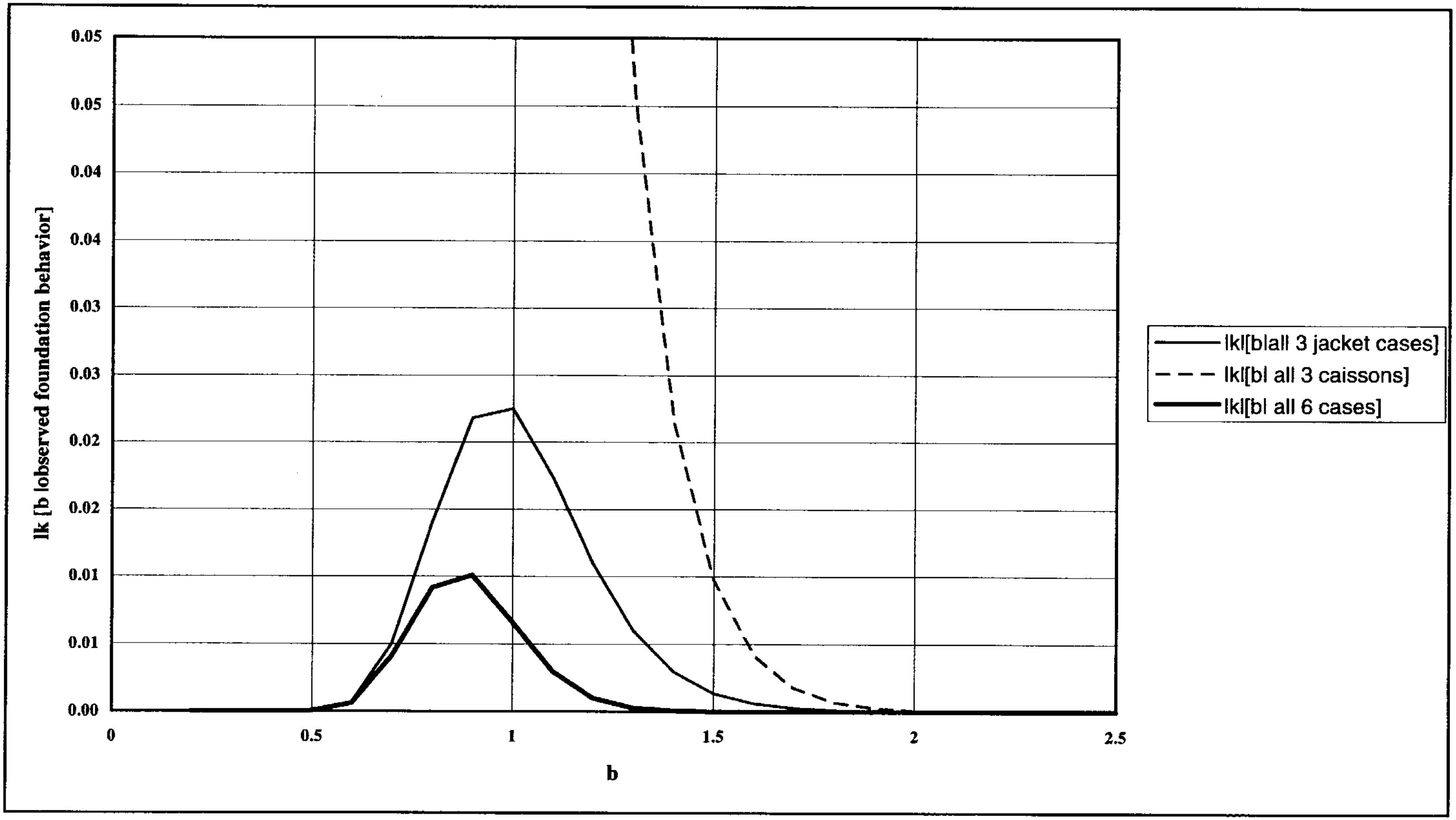


Figure 4-5(b): Group Likelihood Functions - Foundation Lateral Capacity Case C

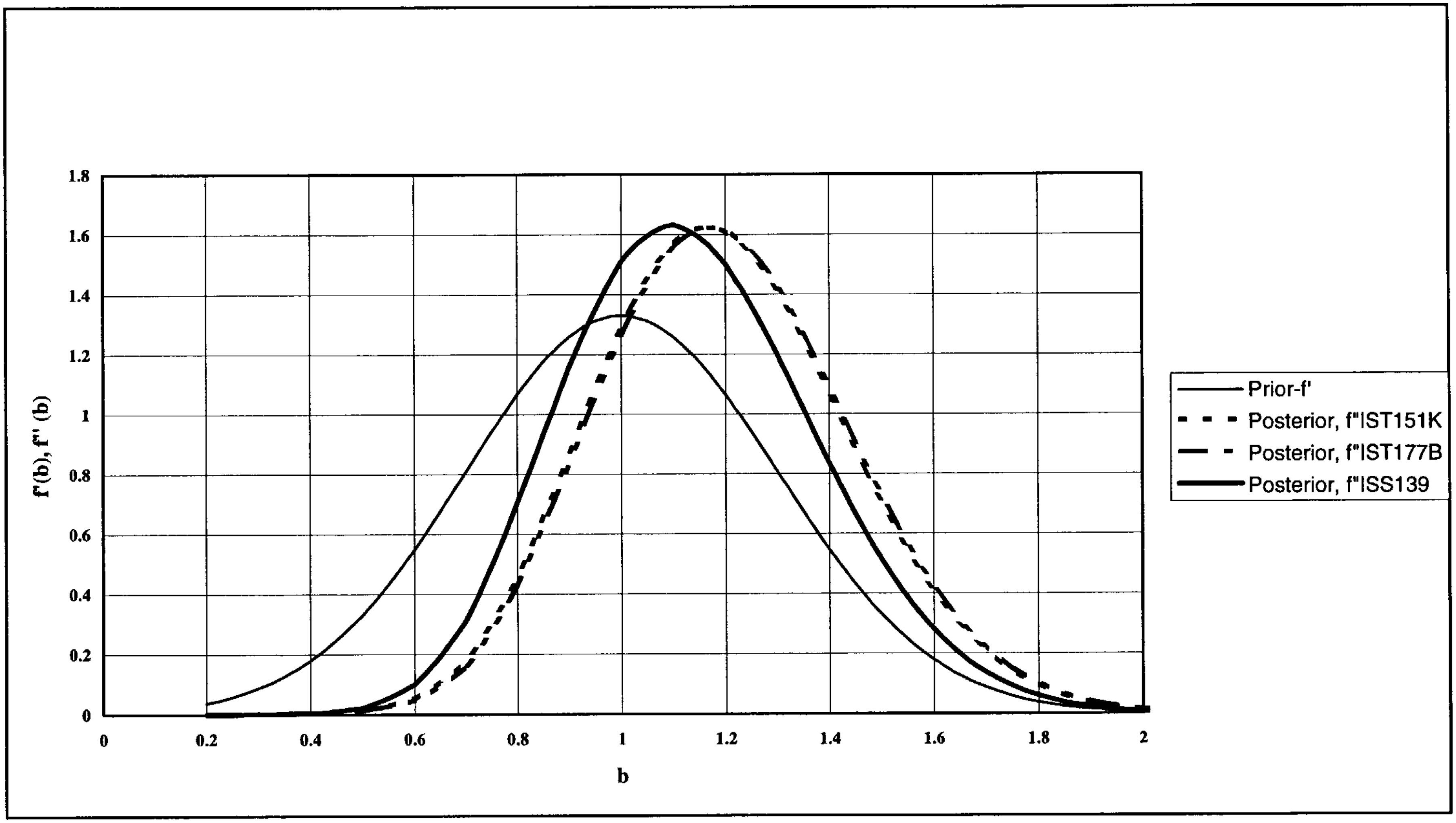


Figure 4-6(a): Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case A Individual Jacket Platforms

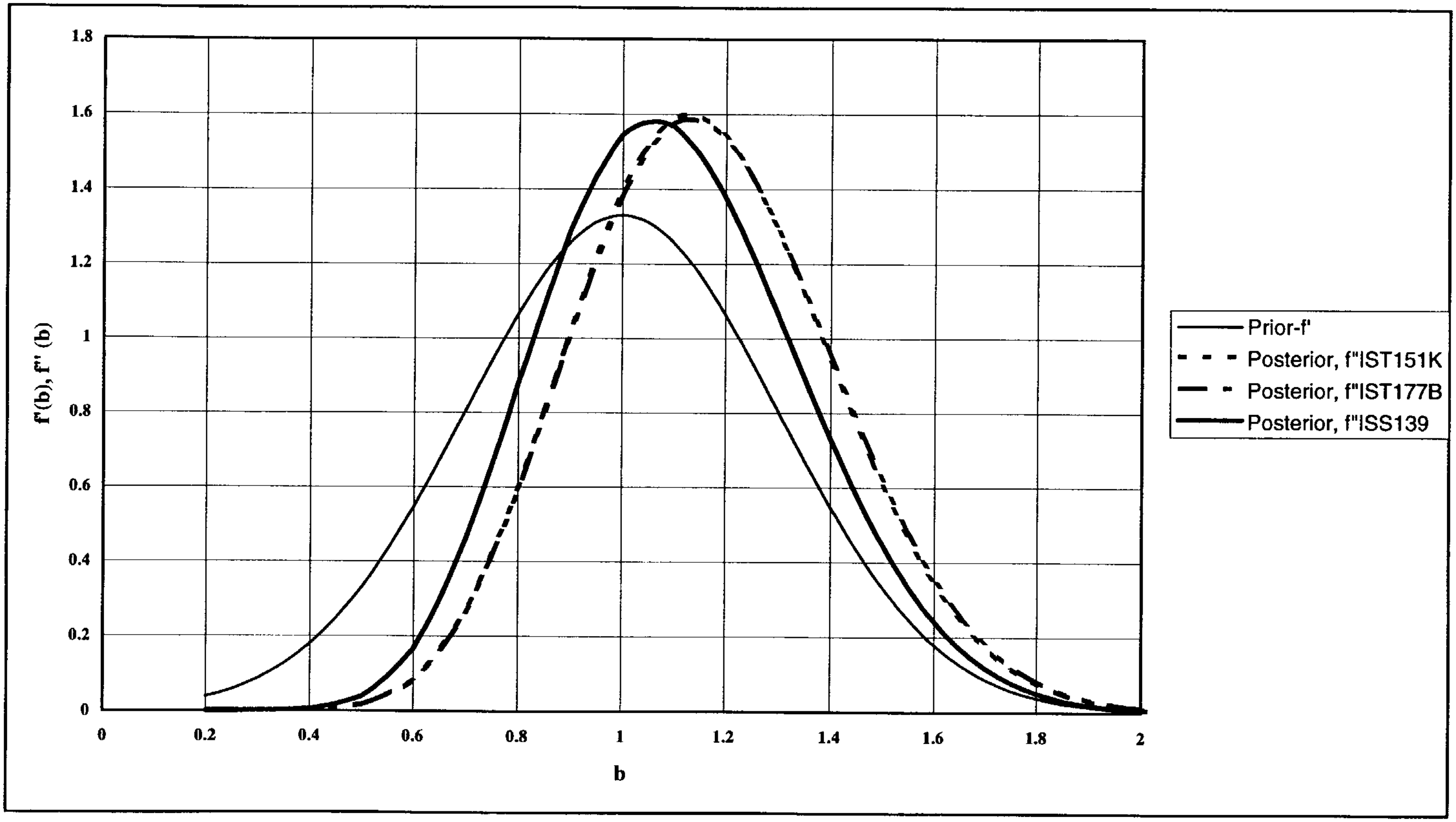


Figure 4-6(b): Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case B Individual Jacket Platforms

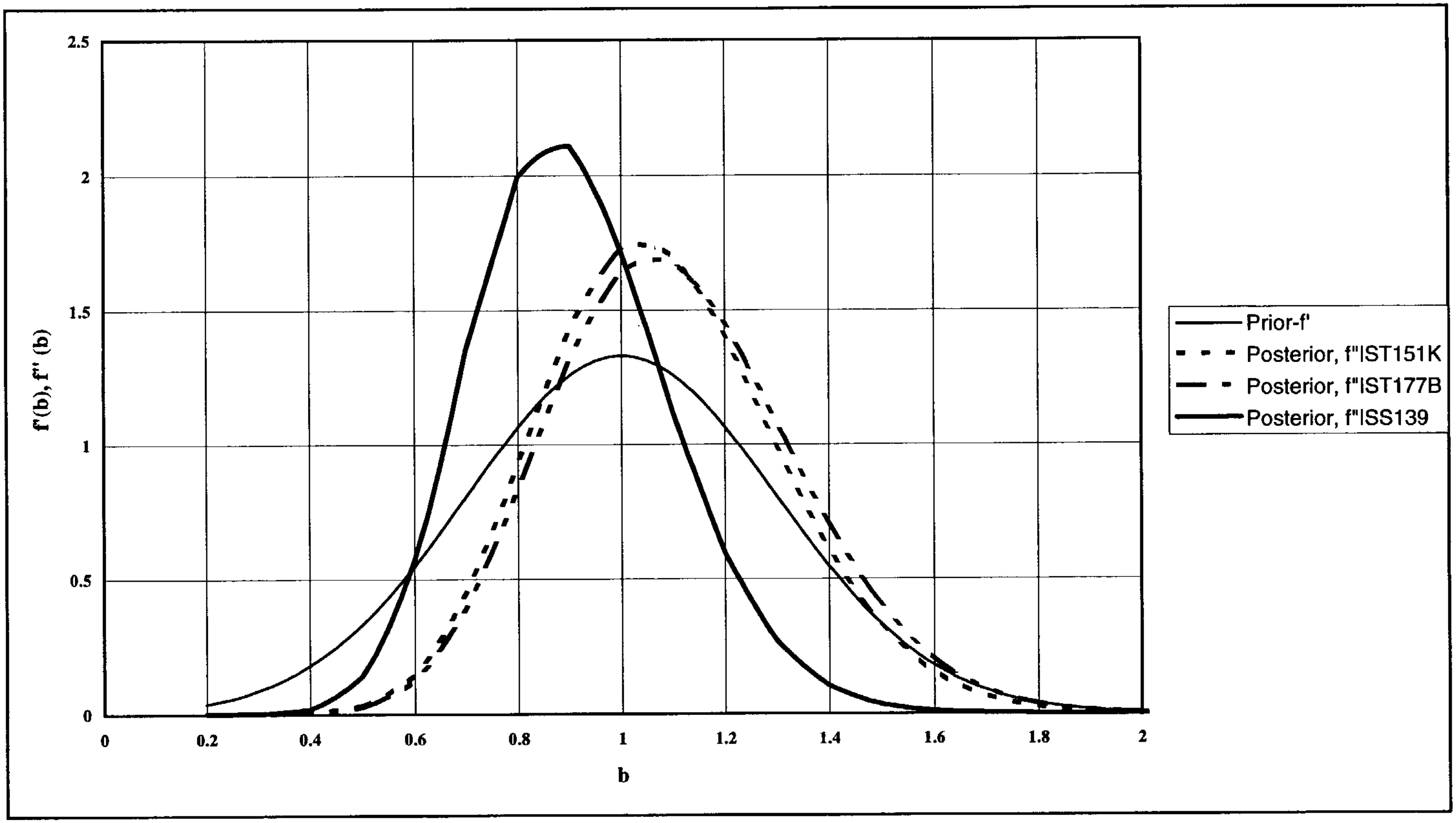


Figure 4-6(c): Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case C Individual Jacket Platforms

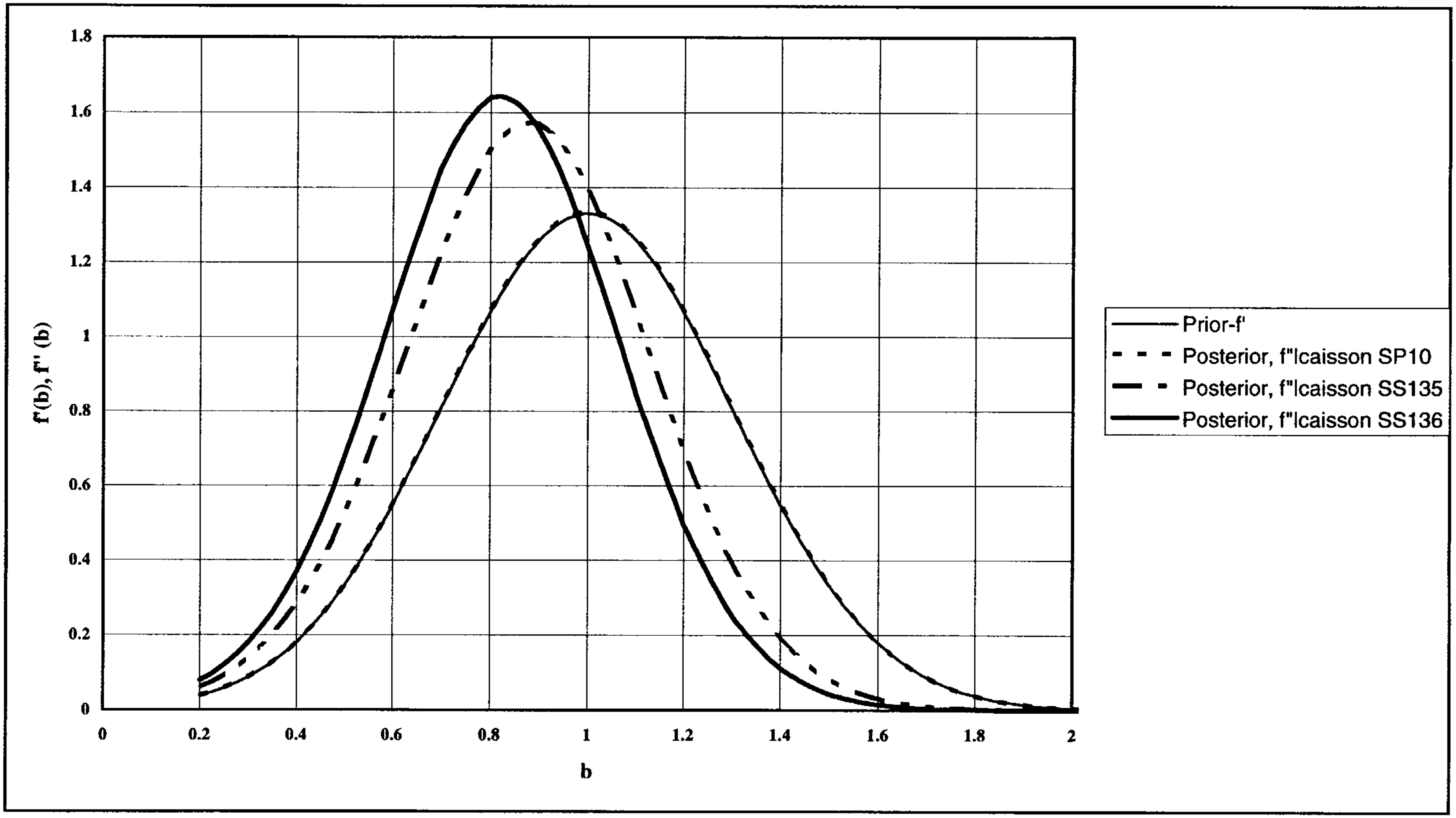


Figure 4-7: Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Individual Caissons

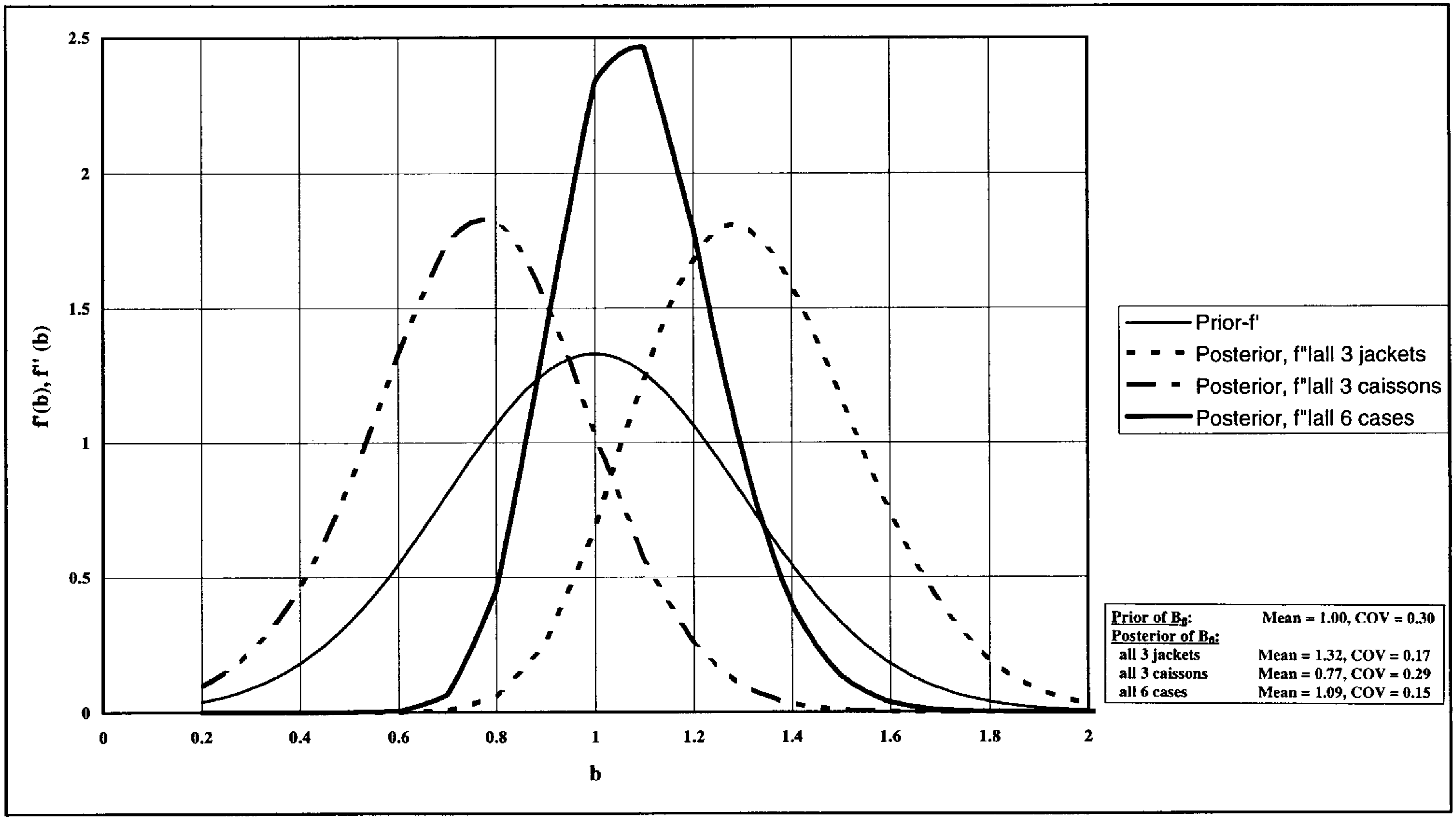


Figure 4-8(a): Posterior Distributions: Lateral Capacity Bias Factor (B_η) - Case A Combined Effect of Jackets and Caissons

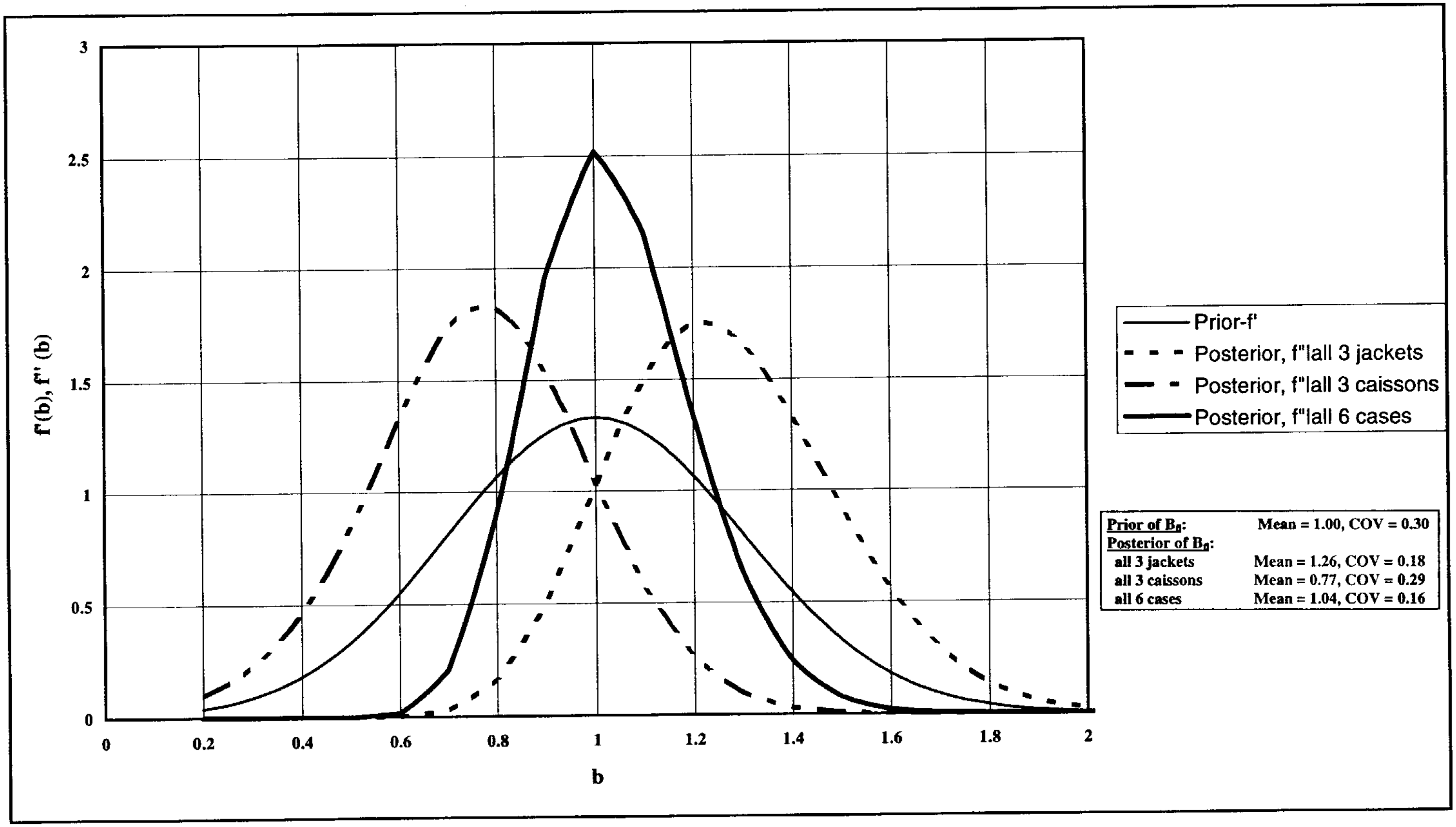


Figure 4-8(b): Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case B Combined Effect of Jackets and Caissons

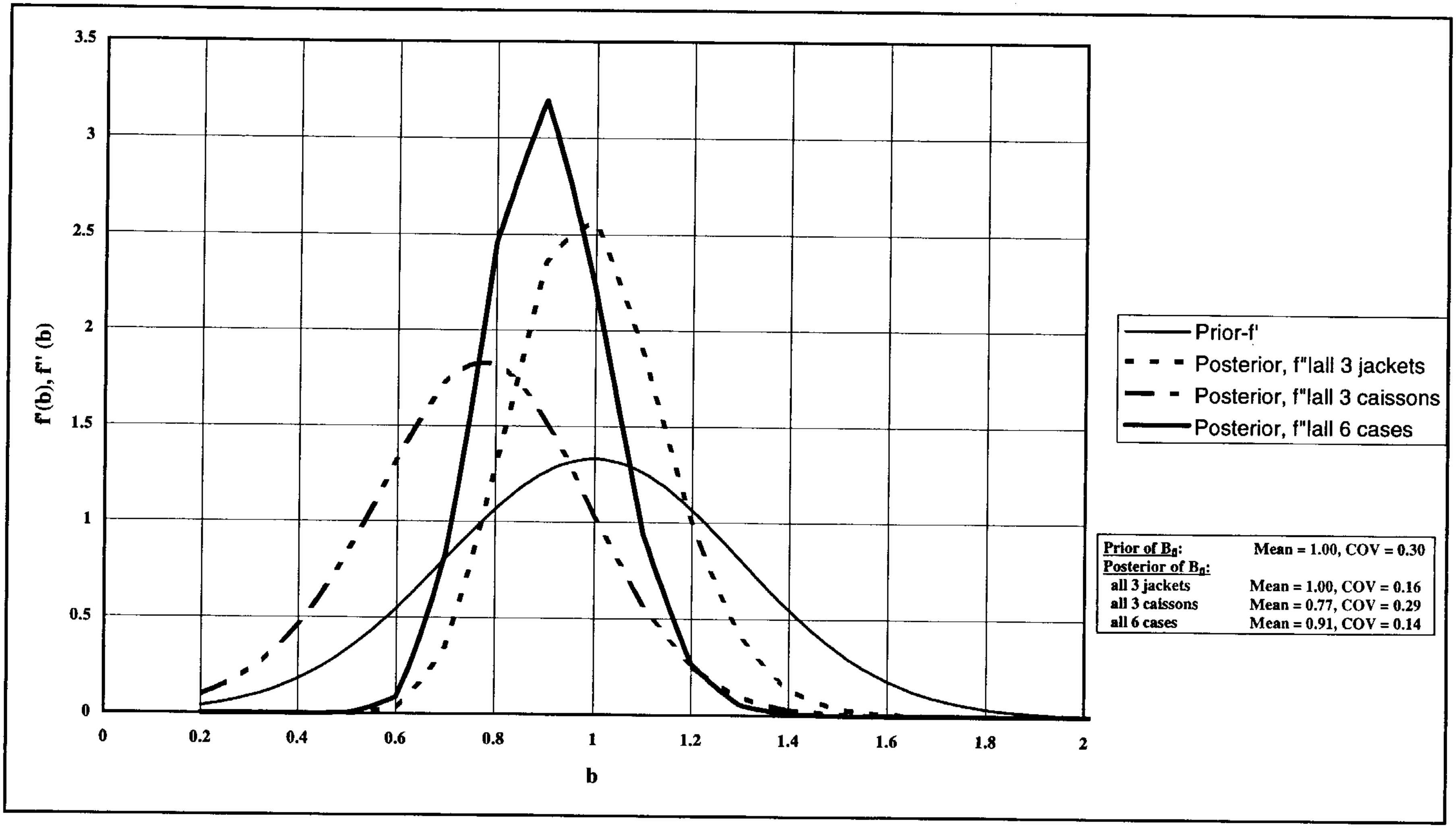


Figure 4-8(c): Posterior Distributions: Lateral Capacity Bias Factor (B_{fl}) - Case C Combined Effect of Jackets and Caissons

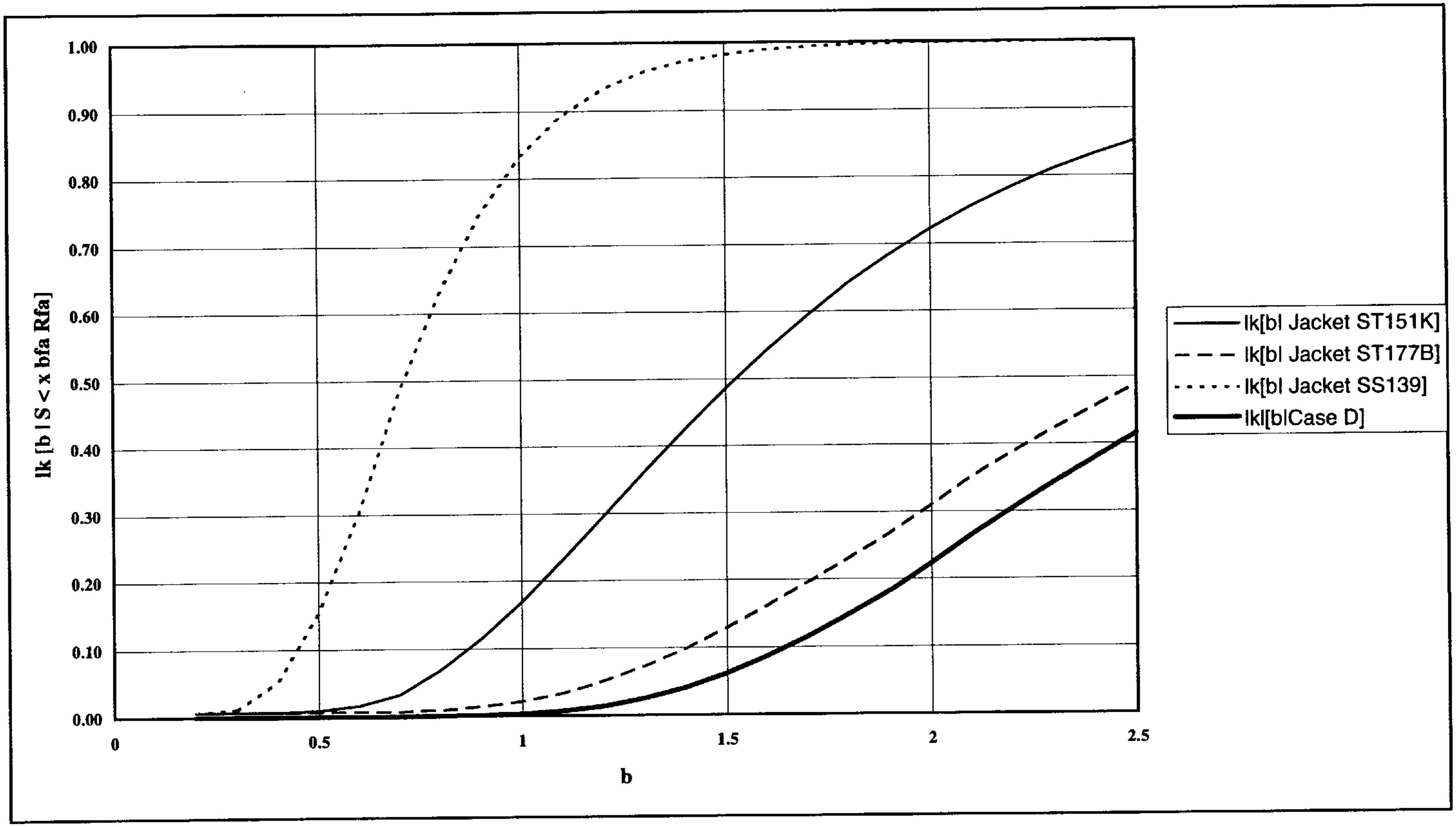


Figure 4-9(a): Likelihood Functions: Foundation Axial Capacity - Case D

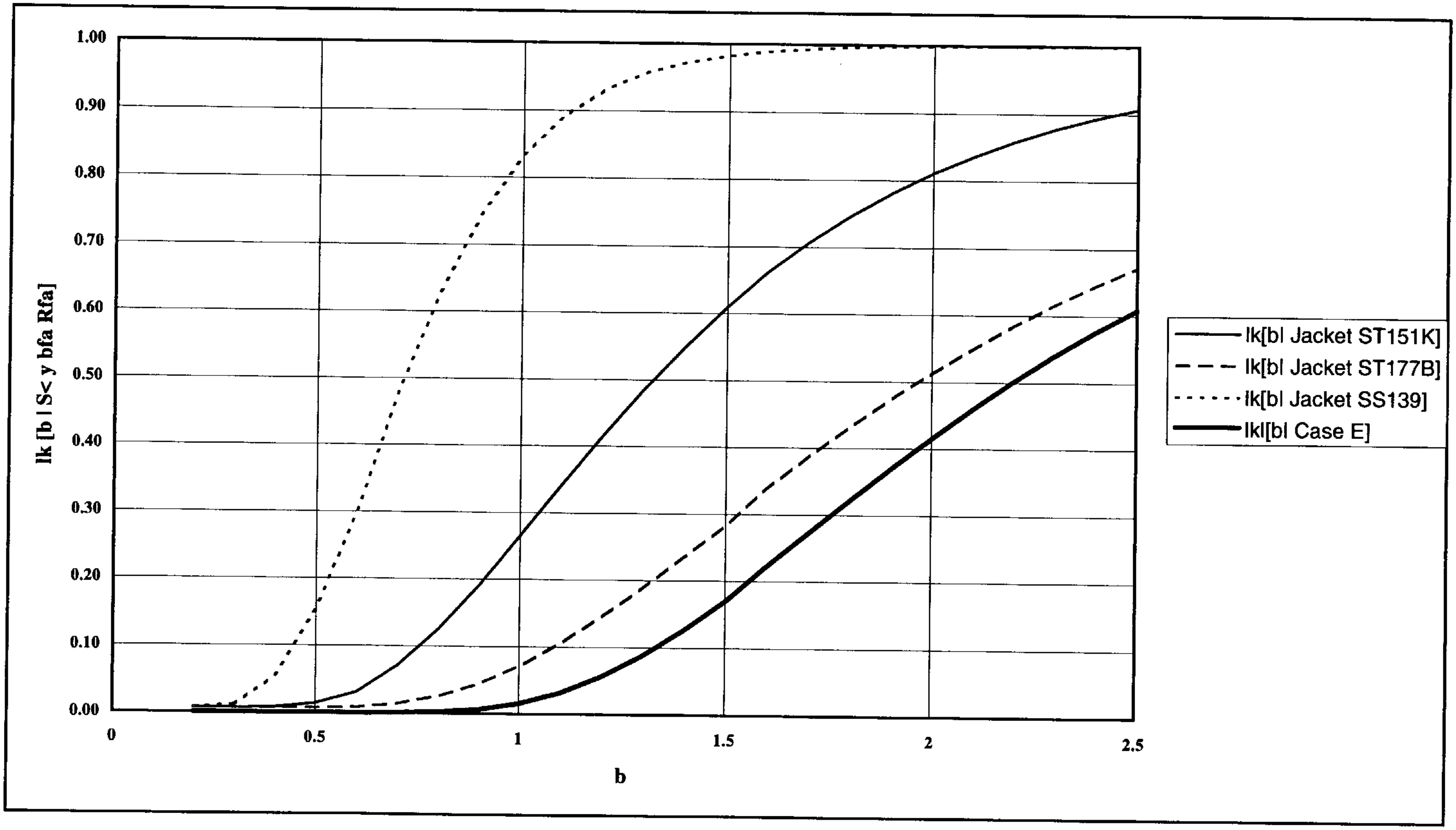


Figure 4-9(b): Likelihood Functions: Foundation Axial Capacity - Case E

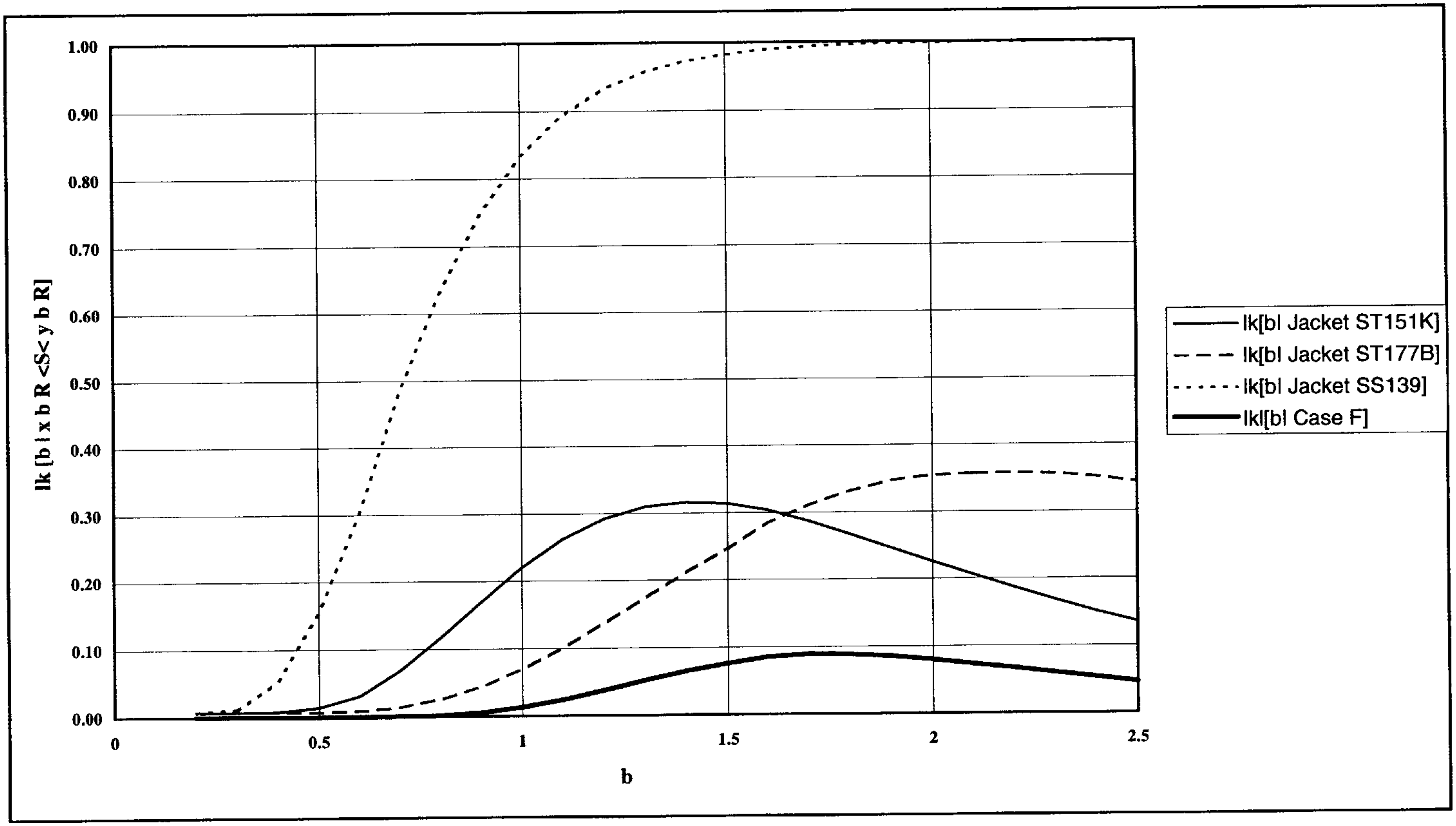


Figure 4-9(c): Likelihood Functions: Foundation Axial Capacity - Case F

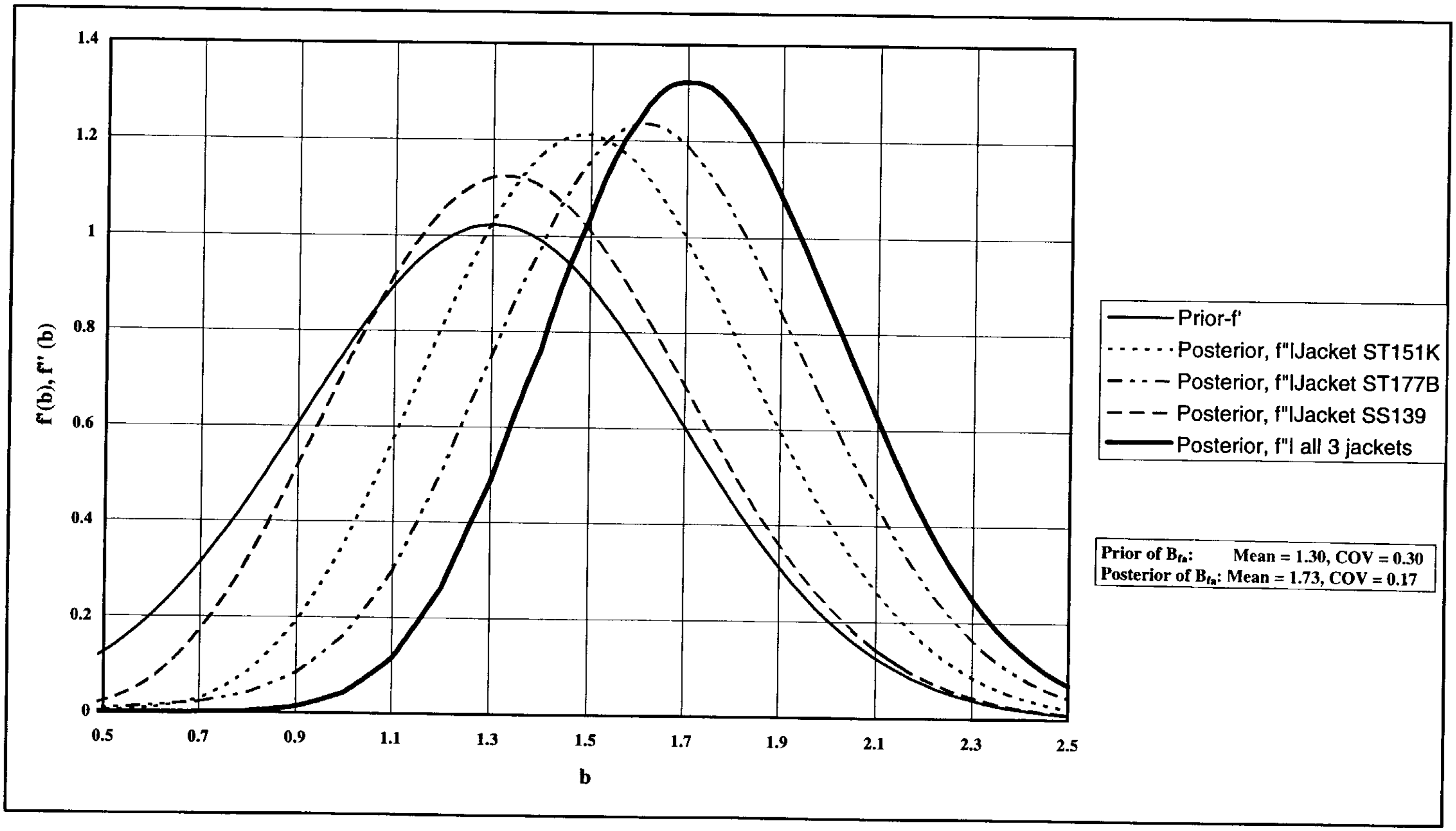


Figure 4-10(a): Posterior Distributions: Axial Capacity Bias Factor (B_{fa}) - Case D

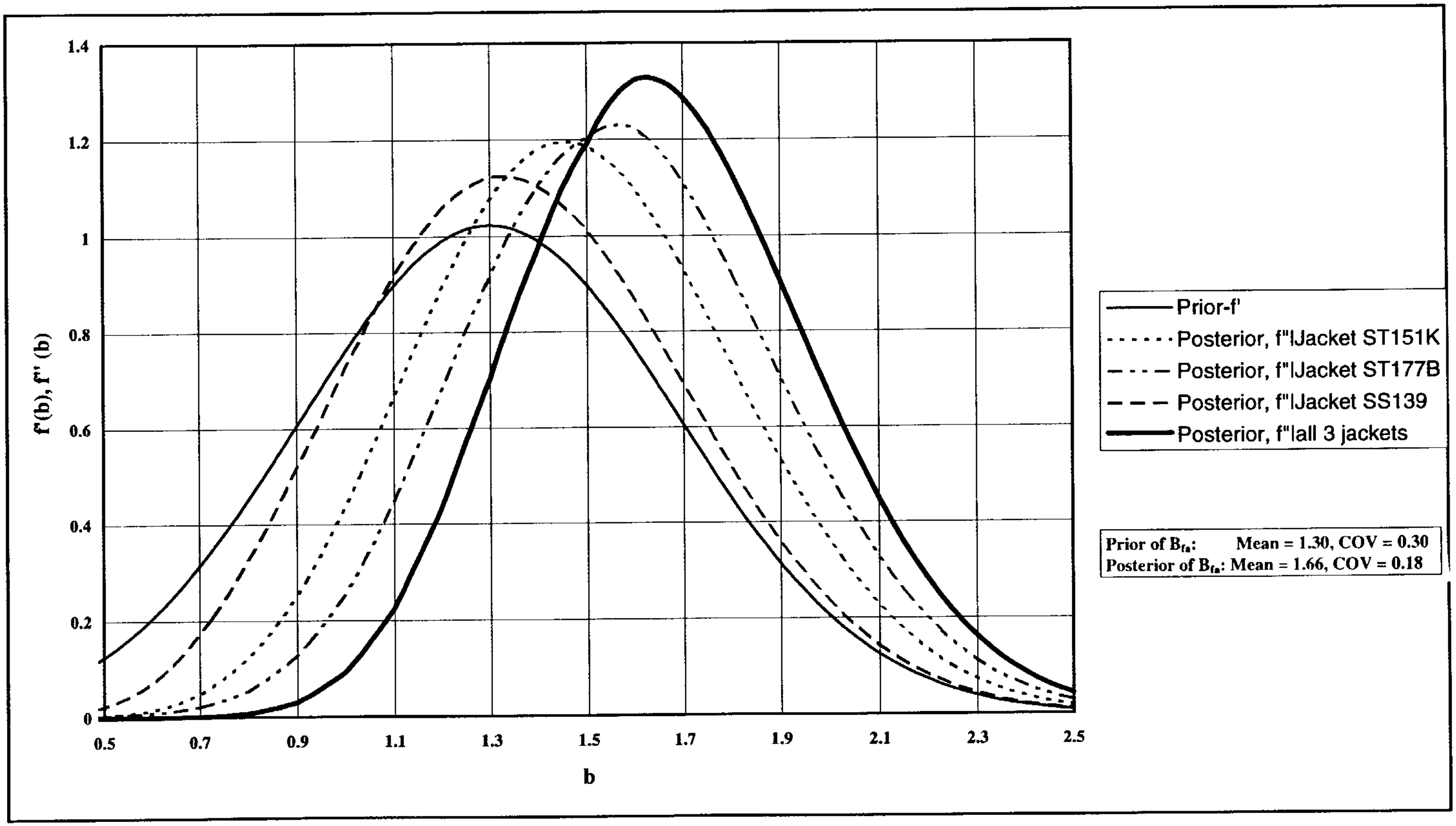


Figure 4-10(b): Posterior Distributions: Axial Capacity Bias Factor (B_{fa}) - Case E

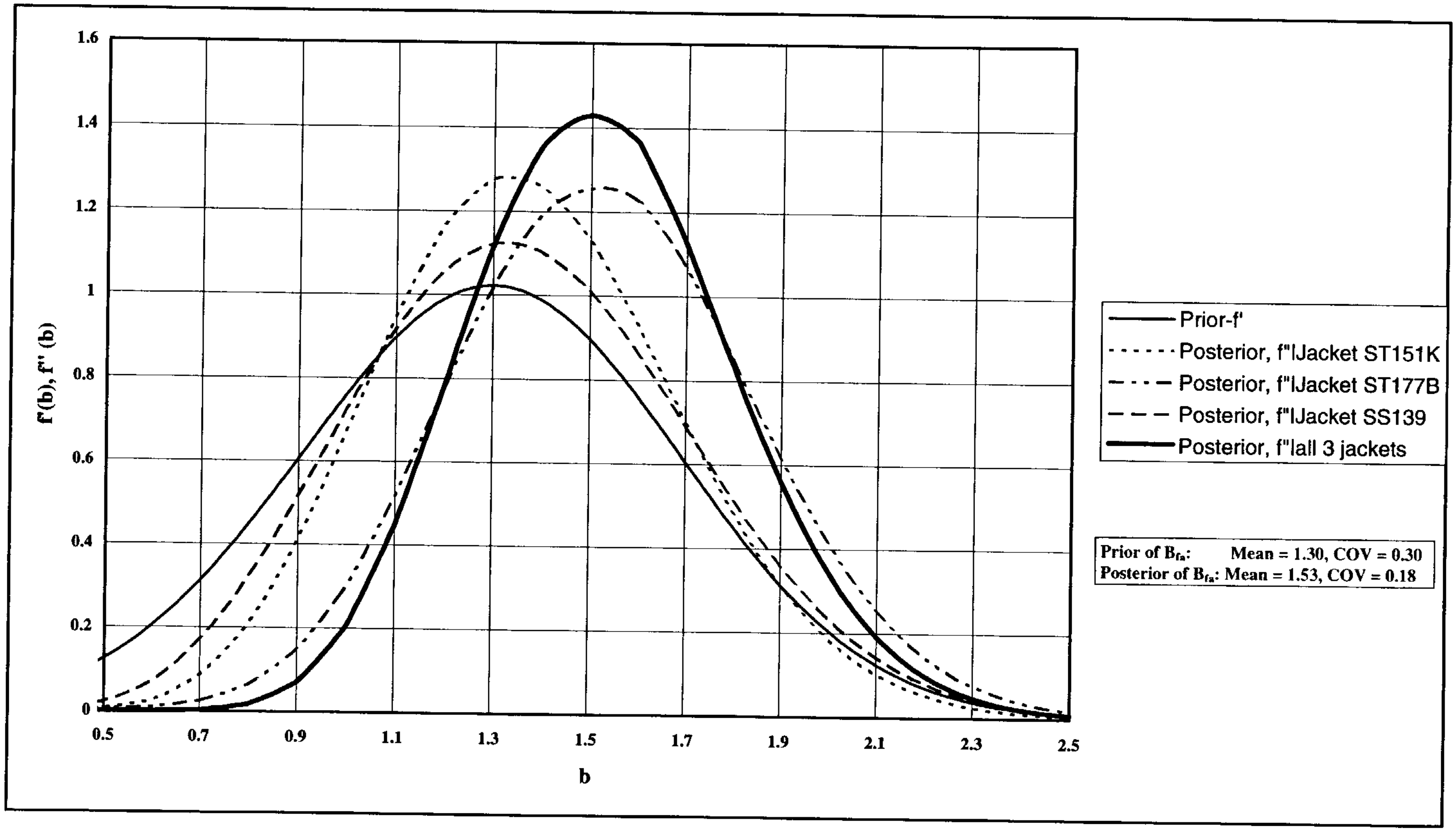
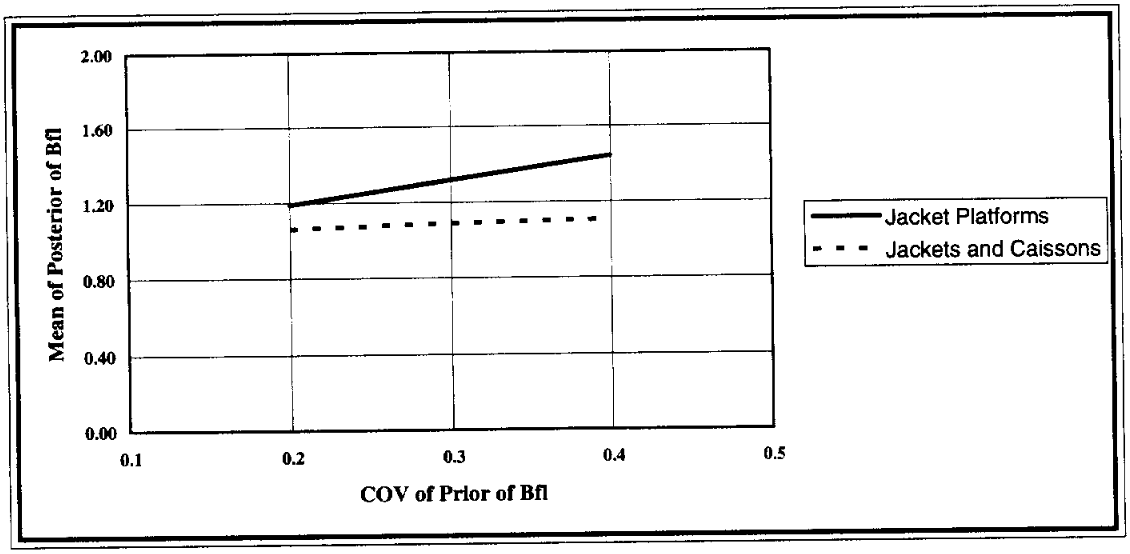
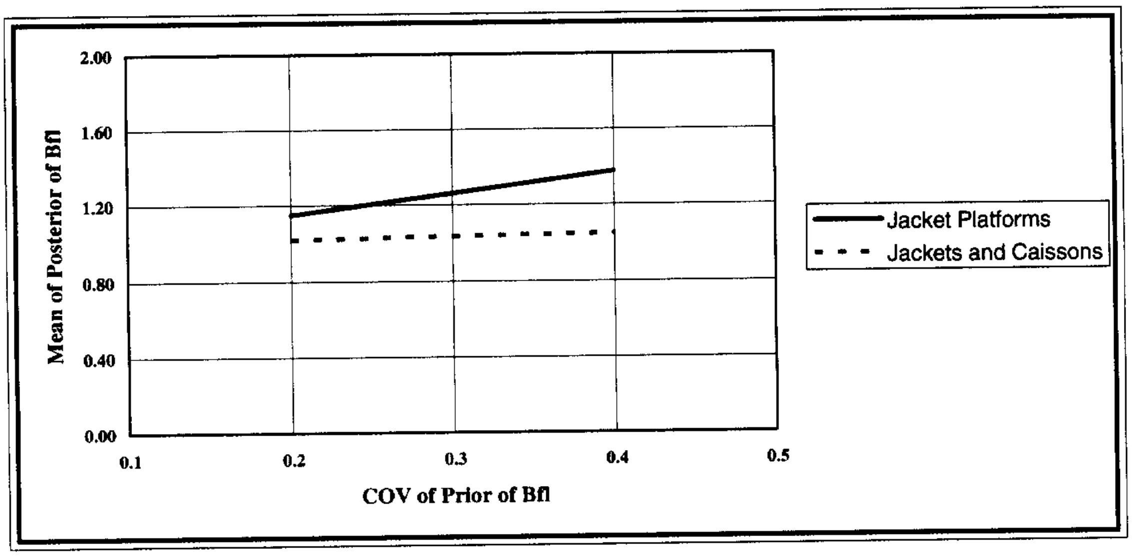


Figure 4-10(c): Posterior Distributions: Axial Capacity Bias Factor (B_{fa}) - Case F

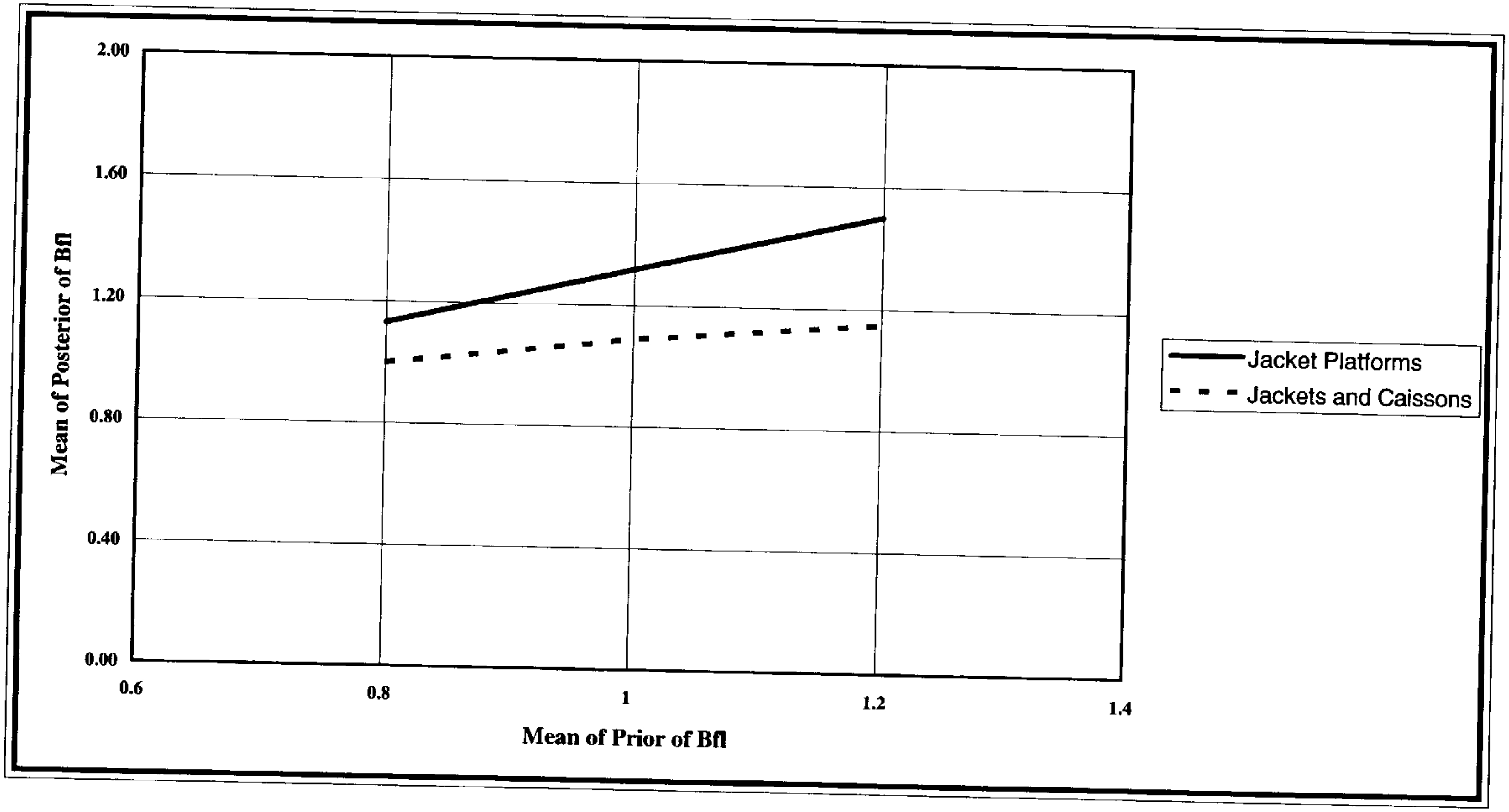


(a): Calibration Case-A **Mean of Prior of Bfl = 1.0**



(b): Calibration Case-B **Mean of Prior of Bfl = 1.0**

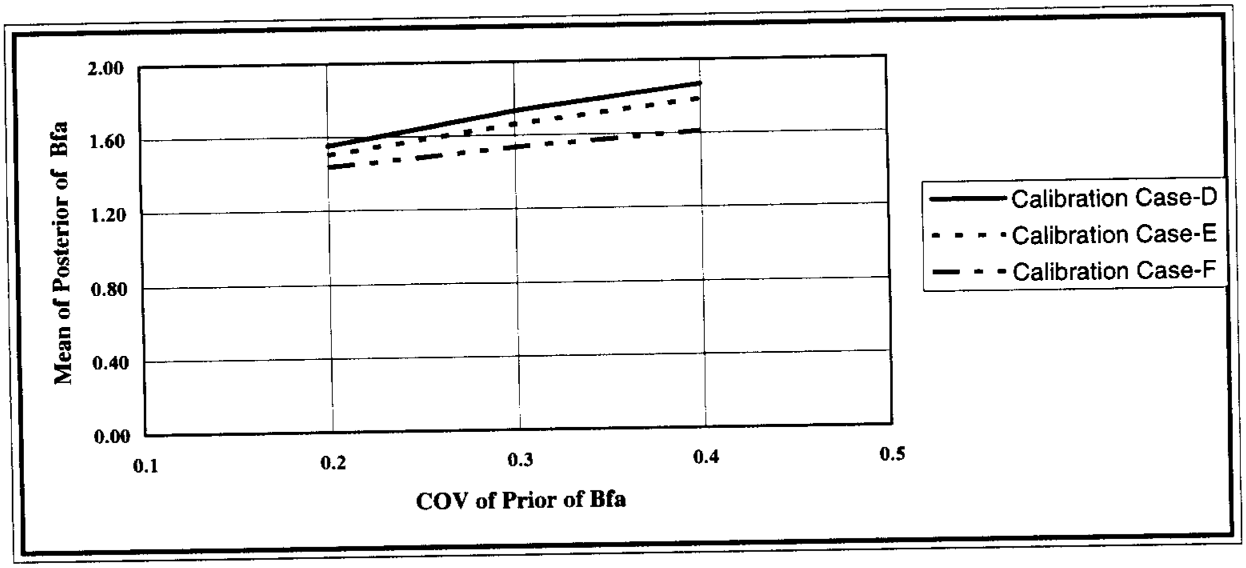
Figure 4-11: Sensitivity of COV of Prior of Bfl on Posterior of Bfl



Calibration Case-A

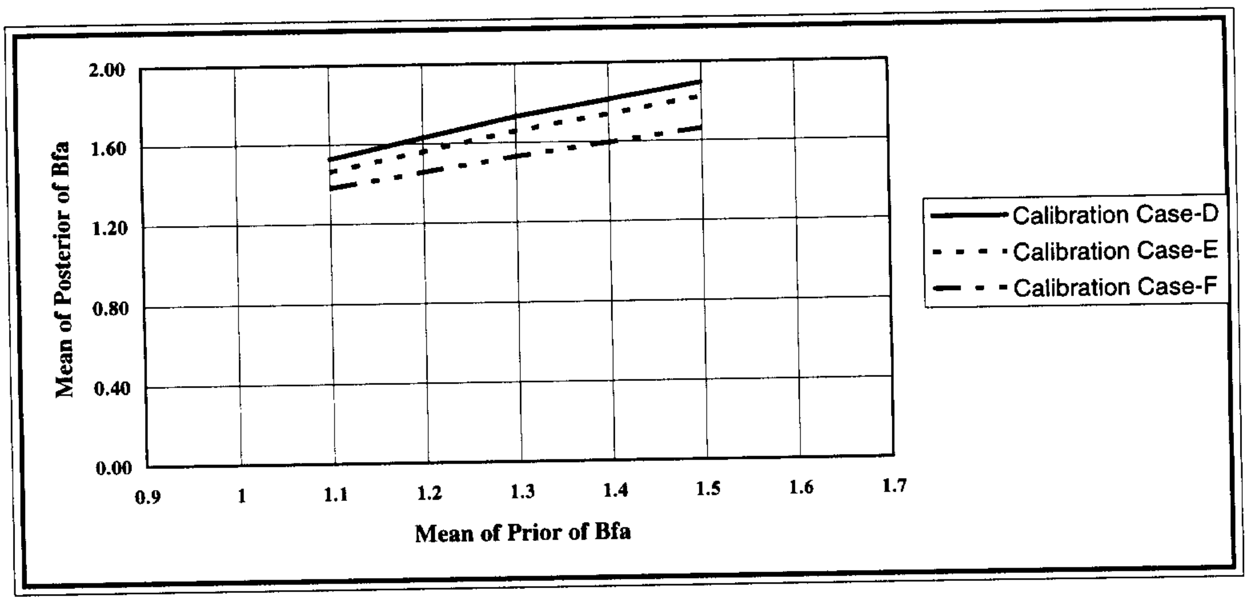
COV of Prior of Bfl = 0.3

Figure 4-12: Sensitivity of Mean of Prior of Bfl on Posterior of Bfl



Mean of Prior of Bfa = 1.3

Figure 4-13(a): Sensitivity of COV of Prior of Bfa on Posterior of Bfa



COV of Prior of Bfa = 0.3

Figure 4-13(b): Sensitivity of Mean of Prior of Bfa on Posterior of Bfa

Table 4-1: Summary of Calibration Inputs and Results - Lateral Capacity (Jacket Platforms)

Platform	Results from Capacity (Pushover) Analysis				Calibration Basis	Posterior of Bfl (#1) for Individual Platforms		Posterior of Bfl (#1) for All 3 Platforms	
	Hindcast Base Shear (Sandrew) Sa (kips)	Ultimate Capacity for Base Case Non-linear Model (Rufl) (kips)	Load Level at First Pile w/Fully Plastic Section Event (RI3) (kips)	Load Level at Multiple Pile Events per Linear Jacket Model (Rfl) (kips)		Mean Value of Bfl	COV of Bfl	Mean Value of Bfl	COV of Bfl
Case-A: Full Plasticity Event in the First Pile did not Occur:									
ST151K	4,860	5,000	4,340	-	S < 4,340 bfl	1.19	0.21	1.32	0.17
ST177B	4,460	5,100	4,330	-	S < 4,330 bfl	1.20	0.21		
SS139 (T25)	1,770	2,410	2,100	-	S < 2,100 bfl	1.14	0.22		
Case-B: Full Plasticity Events in Several Piles did not Occur:									
ST151K	4,860	5,000	-	5,000	S < 5,000 bfl	1.16	0.21	1.26	0.18
ST177B	4,460	5,100	-	5,100	S < 5,100 bfl	1.17	0.21		
SS139 (T25)	1,770	2,410	-	2,400	S < 2,400 bfl	1.10	0.23		
Case-C: Full Plasticity Event in One Pile did Occur:									
ST151K	4,860	5,000	4,340	5,000	4,340 bfl < S < 5,000 bfl	1.08	0.21	1.00	0.16
ST177B	4,460	5,100	4,330	5,100	4,330 bfl < S < 5,100 bfl	1.10	0.22		
SS139 (T25)	1,770	2,410	2,100	2,400	2,100 bfl < S < 2,400 bfl	0.90	0.21		

Notes: (#1) - Prior Distribution of Bfl Assumed with Mean of 1.0 and COV 0.30

Table 4-2: Summary of Calibration Inputs and Results - Lateral Capacity (Caissons)

Platform	Results from Capacity (Pushover) Analysis			Calibration Basis	Posterior of Bfl (#1) for Individual Caisson		Posterior of Bfl (#1) for All 3 Caissons	
	Hindcast Base Shear S-Andrew (kips)	Ultimate Capacity for Base Case Non-linear Model (Ru) (kips)	Load Level at Fully Plastic Section Event in Caisson Rp-1 (kips)		Mean Value of Bfl	COV of Bfl	Mean Value of Bfl	COV of Bfl
Case that the Full Plasticity Event in the Caisson Section did Occur:								
SPelto 10	65	48	44	S > 44 bfl	1.00	0.30	0.77	0.29
SS135	132	148	128	S > 128 bfl	0.88	0.29		
SS136	126	181	153	S > 153 bfl	0.83	0.29		

Notes: (#1) - Prior Distribution of Bfl Assumed with Mean of 1.0 and COV 0.30

Table 4-3: Summary of Calibration Results - Lateral Capacity

Calibration Case	Posterior of Bfl - Lateral Capacity (#1)	
	Mean Value of Bfl	COV of Bfl
Case-A: Full Plasticity Event in the First Pile did not Occur:		
3 Jacket Cases Only	1.32	0.17
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	1.09	0.15
Case-B: Full Plasticity Events in Several Piles did not Occur:		
3 Jacket Cases Only	1.26	0.18
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	1.04	0.16
Case-C: Full Plasticity Event in One Pile did Occur:		
3 Jacket Cases Only	1.00	0.16
3 Caisson Cases Only	0.77	0.29
Combined Effect of All 6 Jackets and Caissons	0.91	0.14

Notes: (#1) - Prior Distribution of Bfl Assumed with Mean of 1.0 and COV 0.30

Table 4-4: Summary of Calibration Inputs and Results - Axial Capacity (Jacket Platforms)

Platform	Results from Capacity (Pushover) Analysis				Calibration Basis	Posterior of Bfa (#1) for Individual Platforms		Posterior of Bfa (#1) for All 3 Platforms	
	Hindcast Base Shear (S-Andrew) Sa (kips)	Ultimate Capacity for Base Case Non-linear Model Rufa (kips)	Load Level at First Pile w/Pullout or Plunging Event Rl2 (kips)	Load Level at Multiple Pile Events per Linear Jacket Model Rfa (kips)		Mean Value of Bfa	COV of Bfa	Mean Value of Bfa	COV of Bfa
Case-D: Pullout/Plunging Event for the First Pile did not Occur:									
ST151K	4,860	4,070	3,330	-	S < 3,300 bfa	1.52	0.21	1.73	0.17
ST177B	4,460	3,200	2,300	-	S < 2,300 bfa	1.63	0.20		
SS139 (T25)	1,770	2,500	2,500	-	S < 2,500 bfa	1.37	0.25		
Case-E: Pullout/Plunging Events in Several Piles did not Occur:									
ST151K	4,860	4,070	-	4,000	S < 4,000 bfa	1.49	0.22	1.66	0.18
ST177B	4,460	3,200	-	2,970	S < 2,970 bfa	1.57	0.20		
SS139 (T25)	1,770	2,500	-	2,500	S < 2,500 bfa	1.37	0.25		
Case-F: Pullout/Plunging Event in One Pile did Occur:									
ST151K	4,860	4,070	3,330	4,000	3,300 bfa < S < 4,000 bfa	1.38	0.22	1.53	0.18
ST177B	4,460	3,200	2,300	2,970	2,300 bfa < S < 2,970 bfa	1.53	0.20		
SS139 (T25)	1,770	2,500	2,500	2,500	S = 2,500 bfa	1.37	0.25		

Notes: (#1) - Prior Distribution of Bfa Assumed with Mean of 1.3 and COV 0.30

Table 4-5(a): Summary of Sensitivity of Variations in Prior of Bfl on Posterior of Bfl

Sensitivity Case	Posterior Parameter	Posterior of Bfl (Mean of Prior of Bfl = 1.0)			Posterior of Bfl (COV of Prior of Bfl = 0.3)		
		Prior COV = 0.2	Prior COV = 0.3	Prior COV = 0.4	Prior Mean = 0.8	Prior Mean = 1.0	Prior Mean = 1.2
Calibration Case-A:							
3 Jacket Cases Only	Mean	1.19	1.32	1.44	**1.13**	1.32	**1.50**
	COV	**0.14**	0.17	0.19	0.15	0.17	0.18
Combined Effect of All 3 Jackets and 3 Caissons	Mean	1.06	1.09	1.11	1.00	1.09	1.15
	COV	0.13	0.15	0.16	0.14	0.15	0.16
Calibration Case-B:							
3 Jacket Cases Only	Mean	1.15	1.26	1.38	-	-	-
	COV	0.14	0.18	**0.21**	-	-	-
Combined Effect of All 3 Jackets and 3 Caissons	Mean	1.02	1.04	1.05	-	-	-
	COV	0.14	0.16	0.17	-	-	-

Table 4-5(b): Summary of Sensitivity of Variations in Prior of Bfa on Posterior of Bfa

Sensitivity Case	Posterior Parameter	Posterior of Bfa (Mean of Prior of Bfa = 1.3)			Posterior of Bfa (COV of Prior of Bfa = 0.3)		
		Prior COV = 0.2	Prior COV = 0.3	Prior COV = 0.4	Prior Mean = 1.1	Prior Mean = 1.3	Prior Mean = 1.5
Calibration Case D	Mean	1.55	1.73	1.86	**1.53**	1.73	**1.89**
	COV	**0.14**	0.17	0.17	**0.17**	0.17	0.16
Calibration Case E	Mean	1.50	1.66	1.78	**1.46**	1.66	**1.82**
	COV	**0.15**	0.18	**0.19**	0.17	0.18	0.17
Calibration Case F	Mean	1.44	1.53	1.61	1.38	1.53	1.66
	COV	0.15	0.18	0.20	0.17	0.18	0.19

Section 5
Conclusions and Recommendations

5.1 GEOTECHNICAL ASPECTS

The predominant soil types at locations of all platforms investigated were very soft to very stiff gray clay. The undrained shear strength profiles were based on the Miniature Vane test data, which in many cases were found close to the interpreted shear strength profiles provided in the soil reports.

The variation in the pile lateral capacity is determined to be on the order of square root of the variation in the undrained shear strength. In addition, only the lateral p-y springs in the upper zone (up to 60 ft below mudline) get fully mobilized and influence the lateral capacity estimates. Thus, the influence of errors in estimates of the undrained shear strength of clay layers (due to unavailability of site-specific soil reports) is more important for the foundation axial capacity estimates than for lateral capacity.

The various effects, which are likely to increase the foundation axial capacity were identified but the procedures to incorporate their effect for different soil conditions are not fully known. Some recent investigations have shown wide differences in the estimates of increase in the pile axial capacities. The most important effects to include are loading rate and cyclic loading effects, which are known to have compensating effects.

The earlier research discussed in Section 2 has indicated that the API RP 2A, pile axial capacity (t-z) estimates underpredict the true ultimate pile axial capacity. However, in this study no change to the API pile axial capacity characterization was included and the combined effect of the various factors which may lead to increased axial capacities will be reflected in the bias factors.

Instead, based on earlier API studies, a higher mean of 1.3 with a COV of 0.3 was considered for the prior distribution of the bias factor (B_{fa}). The increased mean of the prior is considered to provide a reasonable balance between the effects of the loading rate, cyclic degradation, and choice of undrained shear strength of soil.

5.2 CAPACITY ASSESSMENT

The Case 2 and Case 3 analyses presented in Section 3 provided uncoupled estimates of the lateral loads, which define foundation ultimate lateral and axial capacities, by elimination of the failures predicted in the jacket elements in the base case analysis. Thus, the likely effects of the uncertainties associated with the characterization of the jacket element strength and stiffness were eliminated in Case 2 and Case 3 analysis with the linear jacket and nonlinear foundation models.

For both 8-leg platforms, the pullout and plunging of multiple piles were predicted at very low load levels. These load levels are 18 to 33 percent below the best estimate of the maximum Andrew loading. Whereas, the multiple plastic section events were predicted at load levels up to 14 percent higher than the predicted maximum Andrew load level for these platforms, when API static p-y curves were used. The analysis predicted very small pilehead displacements even at multiple fully plastic pile section events and at multiple pile pullout/plunging events in both lateral and axial directions. Considering that neither failure mode was observable in the post-Andrew field inspections, these results indicate that the predictions of the ultimate lateral capacity of pile foundations per API static p-y curves may not be overly conservative, whereas the API t-z curves may underpredict the axial capacity of the jacket foundation system.

In the case of ST151K (with no observed damage to the jacket and its foundation) significant differences have been found between the predicted failures to the jacket frames and the field observations following Andrew. It is also noted that the low ultimate capacity estimates may be triggered by pile axial (t-z) capacity estimates, which may be conservative. This is a significant finding suggesting that the ultimate overturning capacity of platforms may be underpredicted in some cases.

It is recommended that additional platforms be investigated to further improve the findings of this study and to further develop the improved guidelines for the ultimate capacity analysis of the steel jacket foundation under extreme storm condition.

5.3 CALIBRATION

The bias factors developed represent the modeling errors associated with the ultimate lateral and axial capacity estimates of the jacket foundation system. These factors are applicable to the overall safety margin (resistance divided by loading effects) for platforms during extreme hurricane loading. The bias factors greater than 1.0 indicate that the current platform ultimate capacity analysis procedures would provide conservative results in the sense that more failures would be predicted than would actually occur during storms.

Bias factors of 1.3 and 1.7 were established for the foundation lateral (B_{fl}) and axial (B_{fa}) computed ratios of the ultimate capacity to loading effects (Ru/S) respectively for the steel jacket platforms. This implies that on an average, for the platforms investigated in this project, there is about 30 percent conservatism in the foundation lateral capacity and about 70 percent conservatism in the foundation axial capacity estimates based on the ultimate capacity analysis "recipe" used by the project. These B's are related to the key capacity analysis recipe items followed in this project, thus any major variations in the recipe would influence them.

These estimates are higher than the overall (system) bias factor (B) of 1.2, determined in the Andrew Phase I. The Andrew Phase I bias factor was not applicable to a specific failure mode (jacket structure or foundation) and thus included failure modes of all type. If the effect of the foundation is eliminated to determine the bias factor associated with the superstructure (B_{ss}), it is likely that the bias factor will be lower than that obtained in the Andrew Phase I.

These bias factors (B_{fl} and B_{fa}) are based on a limited number of the steel jacket platforms investigated in this project and thus should be viewed as initial estimates and be considered with caution. It is important to note that these bias factors are not applicable to the capacity estimates of an individual platform and that these are for consideration by the API and the regulatory bodies in their further considerations for the guideline and criteria development for platform assessment against extreme storms. These factors may be considered in determination of the average failure probability estimates and in the economic risk and cost-benefit studies for a fleet of platforms. The estimated values for the bias factors are consistent with a trend determined by other investigators.

PMB Engineering is currently executing Andrew JIP — Phase II, which includes all platforms investigated in this study and an additional six steel jacket platforms. In the Phase II work, revised Andrew hindcast data and a revised ultimate capacity analysis recipe are being used, which could affect the findings of this project. Andrew Phase II is using a more detailed reliability evaluation and Bayesian updating procedure, which would result in multiple bias factors applicable to both the jacket structure and its pile foundation.

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

- Increase the sample cases and include searching for recorded foundation failure cases. The platforms with observed foundation damages during earlier hurricanes (e.g., Hilda, Betsy, Camille etc.) may be included to provide useful information to the bias factor based on survival cases only.

- Consider using improved site-specific geotechnical information by obtaining new boreholes for platforms.

- Investigate the differences in foundation design and installation practices among the older and newer platforms, and their likely effect on bias factors. This study considered platforms installed in 1960's.

- Promote more extensive instrumentation and monitoring of existing platforms.

Section 6
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Appendix A
Selection of Platforms

The selection of platforms was based on review of the following information (also in Section 1.3) available for these platforms:

- Structural characteristics and details of platforms
- Damage to the platforms
- Geotechnical information in the vicinity of platforms
- Hindcast information (same as for Andrew JIP Phase I)
- Ultimate capacity and Bayesian updating results from previous analyses (Andrew Phase I results)
- New ultimate capacity analysis done before the first project meeting

The results from this evaluation are summarized in Table A-1 and A-2.

Steel Jacket Platforms: Andrew Phase I capacity analysis results predicted that non-linear events in foundation elements would occur for most of the platform cases studied. However, Andrew Phase I results were based on using similar soil properties for many platforms. Thus, the results may vary for site-specific soils. None of the steel jacket platforms (survivals, partially or completely damaged cases) had any observed damage to their foundation for calibration of foundation capacity.

At the first project meeting, results taken from previous analysis were presented for platforms ST151K, ST130Q, ST177B, and ST52(T23). Platforms ST130Q and ST52(T23) were analyzed at the beginning of this project with increased soil properties (undrained shear strength). Soil information is not available in the vicinity of platforms ST130Q and ST130A. The results obtained for platform ST130Q (using the soil information for ST134 borelog) indicated that this platform would provide limited information for foundation capacity calibration.

The results presented at the first project meeting for platform ST52 (using ST52 block) indicated that the first yielding of a pile section would occur at a load level much higher than the Andrew Hindcast load level, thus providing limited information to this project. Therefore, PMB obtained soil information for SS139 block and have selected platform SS139(T25), based on review of the Andrew Phase I results (using different soil information) and comparison of shear strength profiles.

Based upon review of the capacity analysis results obtained in Andrew Phase I and due to incomplete damage information available for the two failure (collapse) cases — ST151H and ST130A — it was considered that these may provide limited information for calibration of the foundation capacity. Therefore, the "partial damage cases" with detailed damage information may provide more useful information compared to the "failure cases."

Therefore based on the available information for steel jacket platforms, platforms ST151K (survival case), SS139(T-25) and ST177B (damage cases) were identified as preferred cases. In case of platform ST177B, the soil information available for the block ST189 will be used, because the borelog is at about 1 mile from the platform location.

Pile makeups for these platforms are given in Figures A-1 to A-3.

Free Standing Caisson Platforms: Soil information is not available for two Company A caissons which survived Andrew and the other two caissons were damaged near or above mudline. Per Company A, the damage cases may represent fatigue failures.

In case of Company A's SS99 caisson, there is a 20 ft. sand layer at the top, which may have caused failure at mudline, but the available caisson design information is not sufficient. There are no drawings available for the actual caisson and Company A mentioned that the penetration for this caisson was probably 59 ft., which differs significantly from the 100 ft. provided in the "standard" design drawings for this water depth. This caisson, which lacks sufficient design drawings, is thus not preferred.

Company B's caissons are located in South Pelto blocks and the soil information is available for only South Pelto 10. Both of these have the same design but their driveability records indicated that South Pelto 10 was underdriven by 15 ft. South Pelto 10 case was selected, due to availability of soil information and more definitive damage description.

Information for five caissons of Company C was reviewed. The same soil information was used in the original design for three caissons located in the Ship Shoal 113 blocks, which are within 0.6 miles of the soil borelog. Out of these, the caisson with the minimum damage (leaning 5 degree) is identified as a preferred case. The other two caissons in blocks SS135 and SS136 are identified as good candidates. Finally, two out of these three caissons will be selected, based upon review of their site-specific hindcast data, soil shear strength information, and capacity estimates.

Table A-1: Features of Steel Jacket Platforms

Platform Identification	Block Number	Year Installed	Water Depth (ft.)	Number of Legs/Piles	Foundation Features: Pile Diameter (inch)	Foundation Features: Penetration Below Seabed (ft.)	Damage Description (#1)	Availability of Structural Details	Availability of Soil Report	Case Type for Bayesian Updating	Selection for the Foundation JIP
ST 151K	South Timbalier 151	1963	137	8	30 inch	175 ft.	None	Yes	Yes (same block)	Survival case	YES
ST130Q	South Timbalier 130	1964	170	4	30 inch	210 ft.	None	Yes	No (closer to ST134 borelog)	Survival case	(#2)
T23	South Timbalier 52	1969	63	4	36 inch	170 ft.	Tearing of K-joints	Yes	Yes (same block)	Damaged case	(#3)
T25	Ship Shoal 139	1969	62	4	36 inch	165 ft.	Tearing of K-joints (Jacket salvaged)	Yes	Yes (same block)	Damaged case	YES
ST 177B	South Timbalier 177	1965	142	8	30 inch	187 ft.	K-joints, X-joints (Jacket salvaged)	Yes	Yes (ST189 borelog at about 1 mile)	Damaged case	YES (#4)
ST 151H	South Timbalier 151	1964	137	8	30 inch	180 ft.	Rubbled Details not available	Yes	Yes (Same Block)	Collapse case	(#5)
ST 130A	South Timbalier 130	1958	180	8	30 inch	?	Leg failure occurred before pile yield	Yes	No (farther away from ST134)	Collapse case	(#2)

Notes:

#1 Damage observed post hurricane Andrew
#2: Soil report in the same block or in vicinity not available
#3: Preliminary evaluation (presented at the 1st Meeting) showed that it will not give required information
#4: Damaged case with soil borelog at about 1 mile, hence selected
#5: Adequate damage information not available, thus may not provide sufficient information for calibration.

Table A-2: Features of Free Standing Caisson Platforms

Company	Block Number	Year Installed	Water Depth (#1) (ft.)	Structural Features		Damage Description (#2)	Availability of Structural Details	Availability of Soil Report	Case Type for Bayesian Updating	Selection for the Foundation JIP
				Caisson Diameter (inch)	Penetration Below Seabed (ft.)					
A	Ship Shoal 99	1961	21	48 inch	59 ft.	Broken at mudline (may be fatigue damage)	Drawings for particular location not available (#3)	Yes (Same Block)	Damaged case (may not be a foundation failure)	(#5)
	Ship Shoal 98	1986	17	72 inch	75 ft.	No damage	Yes	No	Survival case	(#5)
	South Timbalier 52	1977	60	6 ft. to 8 ft. tapered	103 ft.	Severed 5' above mudline	?	Yes (Same Block)	Damaged Case (not a foundation failure)	(#5)
	South Timbalier 51	1982	55	6 ft. to 8 ft. tapered	93 ft.	No damage	Yes	No	Survival case	(#5)
B	South Pelto 9	1985	35	30 inch	195 ft.	Leaning	Yes	No (in vicinity of SPelto 10)	Damaged/Failure case (Foundation failure)	-
	South Pelto 10	1984	35	30 inch	195 ft.(as designed) 180 ft.(as-installed)	Leaning 12 degree	Yes	Yes (Same Block)	Failure case (Foundation failure)	Yes
C	Ship Shoal 113	1990	46	48 inch	100 ft.	Toppled over	Yes	Yes (#4) (borelog within 0.2 miles)	Failure case (Foundation failure)	-
	Ship Shoal 113	1988	48	48 inch	100 ft.	Leaning 48 degree	Yes	Yes (#4) (borelog within 0.2 miles)	Failure case (Foundation failure)	-
	Ship Shoal 113	1990	47	48 inch	100 ft.	Leaning 5 degree	Yes	Yes (#4) (borelog within 0.6 miles)	Damaged/Failure case (Foundation failure)	Yes (#6)
	Ship Shoal 135	1983	53 (48' original)	48 inch	100 ft.	Leaning 15 degree	Yes	Yes (borelog within 0.8 miles)	Failure case (Foundation failure)	Yes (#6)
	Ship Shoal 136	1983	50	48 inch	100 ft.	Leaning 30 degree	Yes	Yes (Same well location)	Failure case (Foundation failure)	Yes (#6)

Notes:

#1 Where available, the as-is water depth is mentioned.

#2 Damage observed post hurricane Andrew

#3 Penetration for standard design differ

#4: Same soil report used in design

#5: Failure cases are sought to provide an upper bound to the "bias Factor" distribution based on Jacket ca

#6: To select 2 caissons (out of 3) upon review of the site-specific hindcast data and soil shear strength

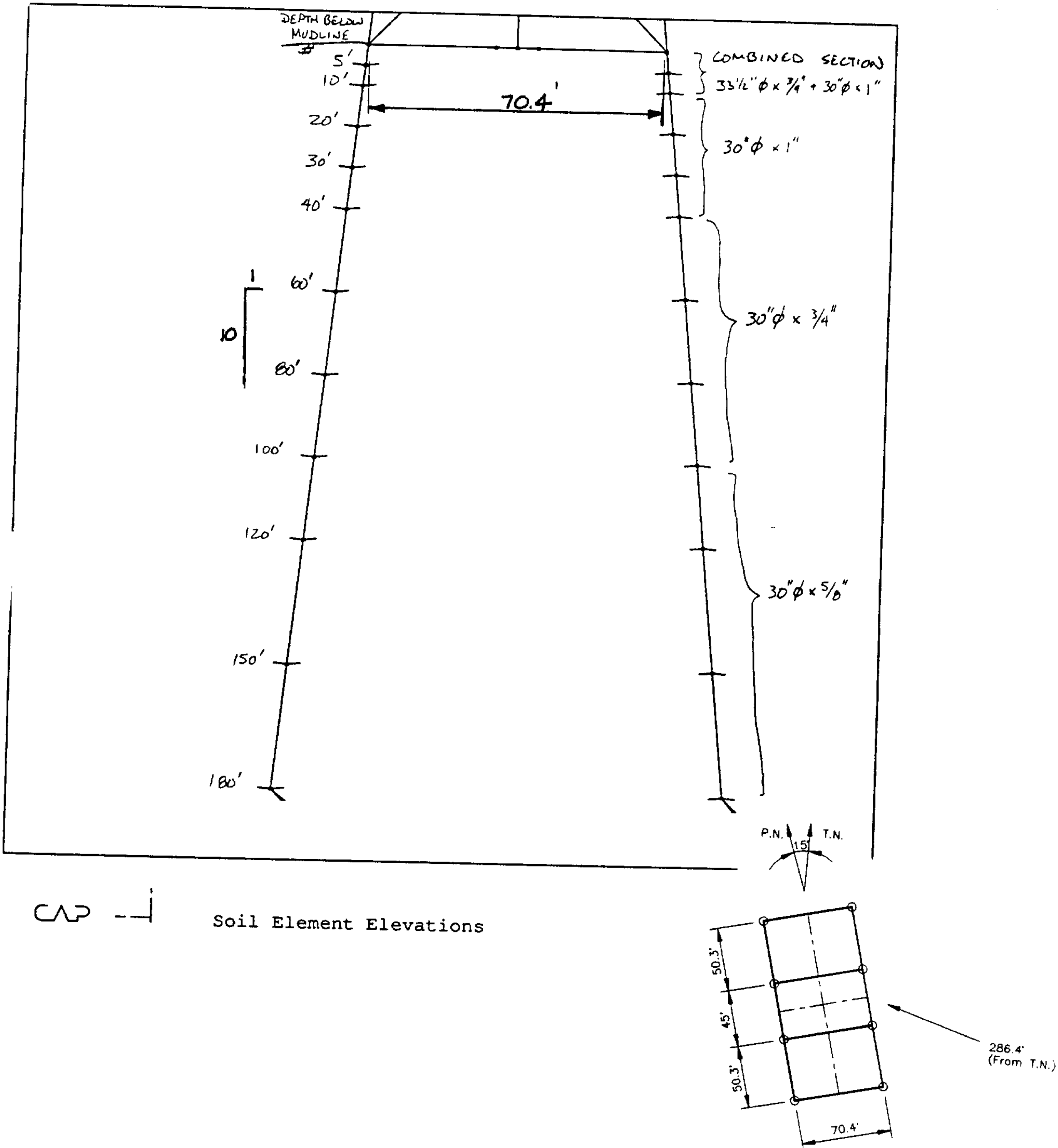


Figure A-1: Pile Makeup - Jacket Platform ST151K

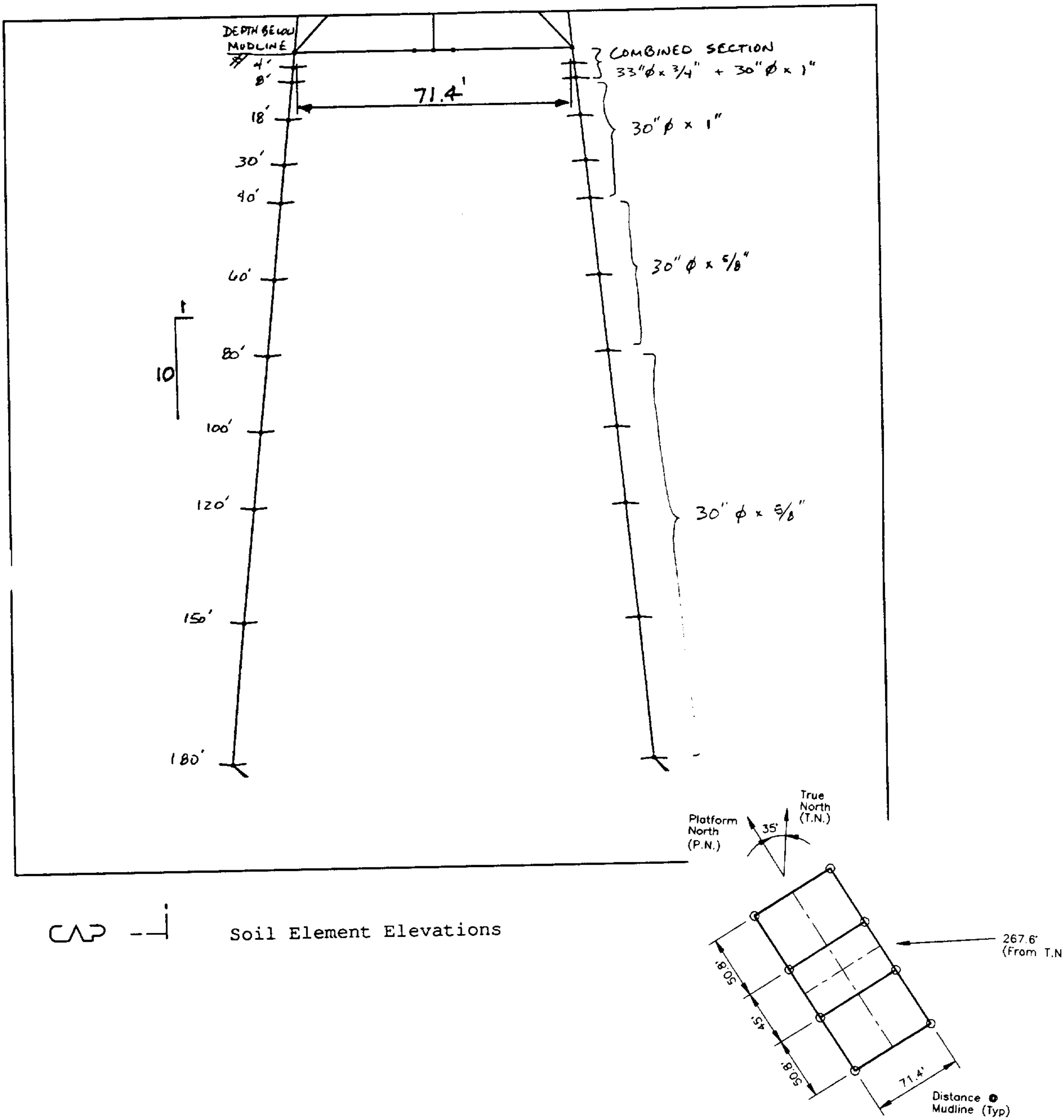


Figure A-2: Pile Makeup - Jacket Platform ST177B

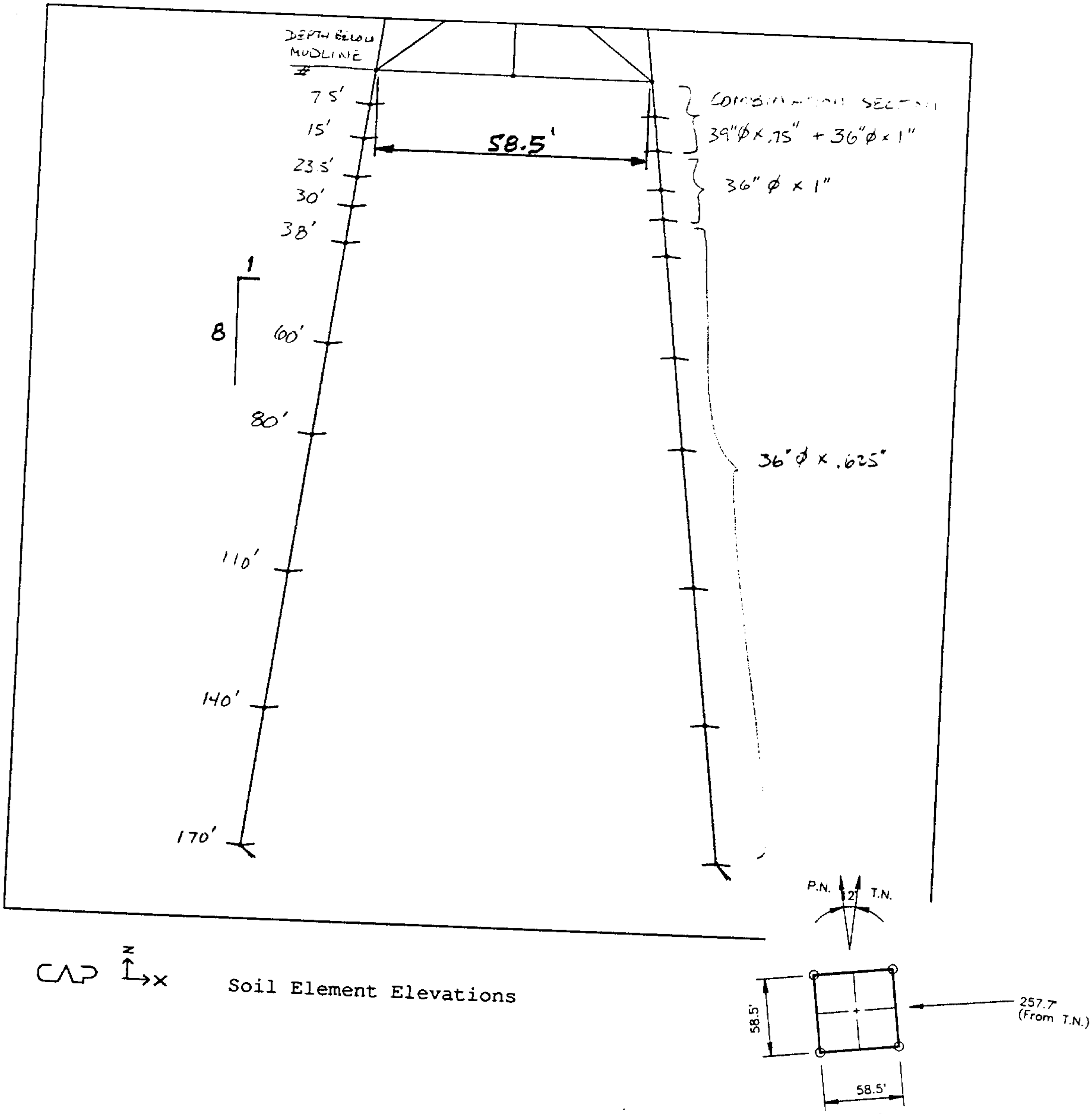


Figure A-3: Pile Makeup - Jacket Platform SS139

Appendix B
Probability of Failure Formulation

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

$$P_f = \int_o^\infty \{1 - F_S(x)\} f_R(x)\, dx \qquad \text{(B-1)}$$

where f_R is the capacity PDF (probability density function) and F_S is the CDF (cumulative distribution function) of load.

Normally the random load, S, represents the maximum load in any one-year period. In this case, the load S is the maximum load on the structure during Andrew and is represented by the random base shear, BS, obtained as follows:

$$BS = C1\,[h + C2\,u]^{C3} \cdot \epsilon_o \qquad \text{(B-2)}$$

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). ϵ_o represents correction factor in base shear estimates, due to wave-to-wave variability, and is assumed to have a log-normal distribution with mean of 1.0 and COV of 0.20.

It is assumed that the probability distribution of each random wave height, H follows the empirical Forristall distribution:

$$f_{H|H_s}\left(h \mid H_{s_j} = h_s\right) = \frac{\alpha 4^{\alpha}}{\beta H_s}\left(\frac{x}{H_s}\right)^{\alpha-1} \exp\left[-\frac{4^{\alpha}}{\beta}\left(\frac{x}{H_s}\right)^{\alpha}\right] \qquad \text{(B-3)}$$

in which $\alpha = 2.126$, $\beta = 8.42$, and H_s is the significant wave height.

Using the probability distribution of H and the formulation of base shear, the final (marginal) CDF for the maximum base shear, F_{MBS}, during the multi-hour (unidirectional) "storm" is obtained as follows:

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

in which F_{BS} is the lognormal cumulative distribution implied by Eq. B-2, Nj denotes the number of random waves in an hour with significant wave heignt, (h_s) and current (u). ϵ_1 and ϵ_2 are the PDF's of the "errors" in the hindcast of the significant wave heights and currents respectively during Andrew. Note that in this equation $h_{sj} = (H_{sj}\, \epsilon_1)$ and $u_j = (U_j\, \epsilon_2)$, where H_{sj} and U_j are the hindcast estimates.

Then the probability of failure can be calculated by numerical integration of Eq. B-1, assuming a lognormal distribution on R, with a specified median and COV. In order to correct for possible bias in the wave force and ultimate capacity procedures a factor B was introduced. Failure therefore is presumed to be associated with $BR/S < 1$ rather than $R/S < 1$. The probability of failure for a given B is obtained as follows:

$$P_f(b) = \int_o^{\infty} \{1 - F_s(bx)\} f_R(x)\, dx \tag{B-5}$$

Appendix C
Soil Shear Strength Profiles and Borelogs

Figure C-1 - Shear Strength Profile - Block ST 151 - Driven Samples

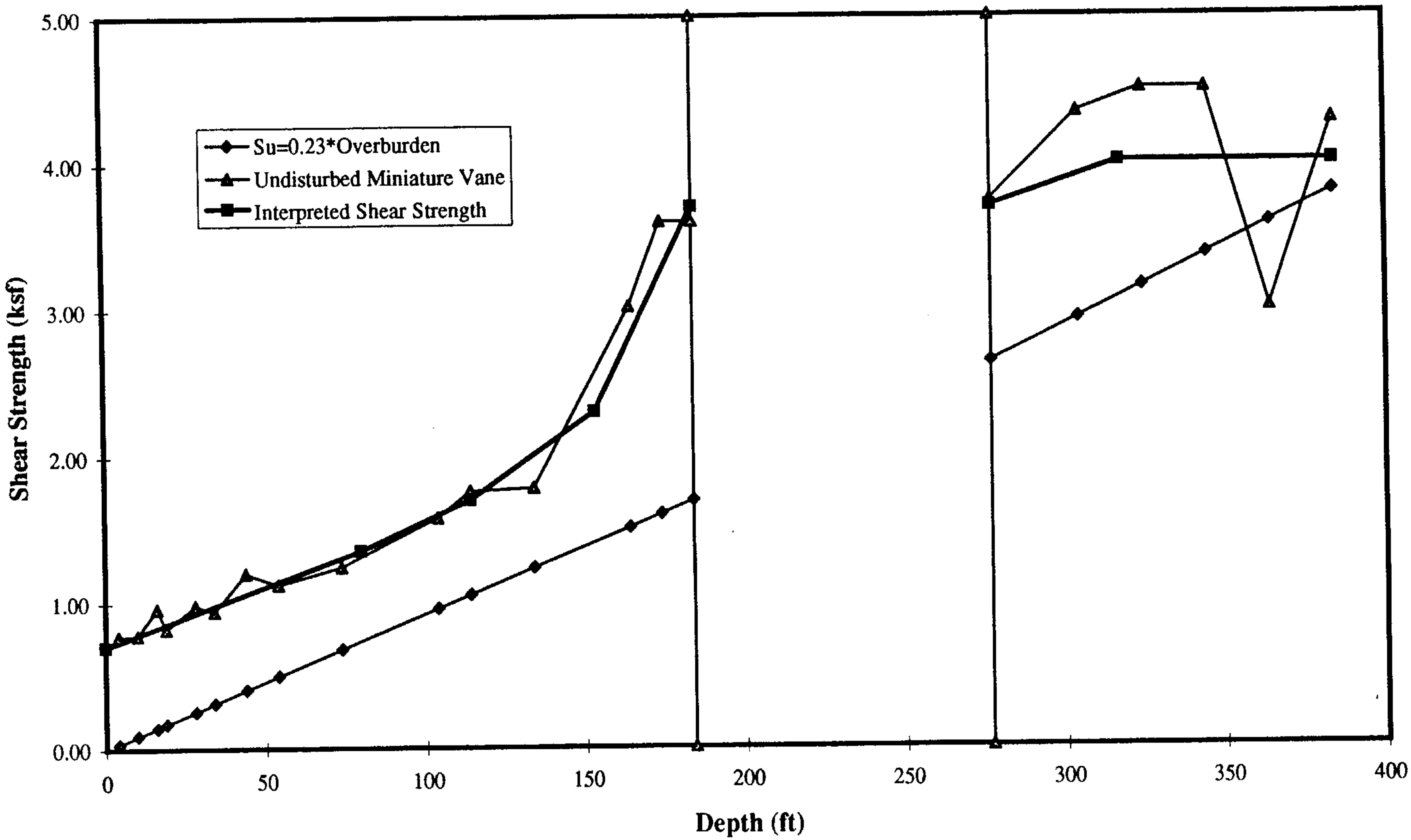


Figure C-2 - Shear Strength Profile - Block ST 189 - Driven Samples

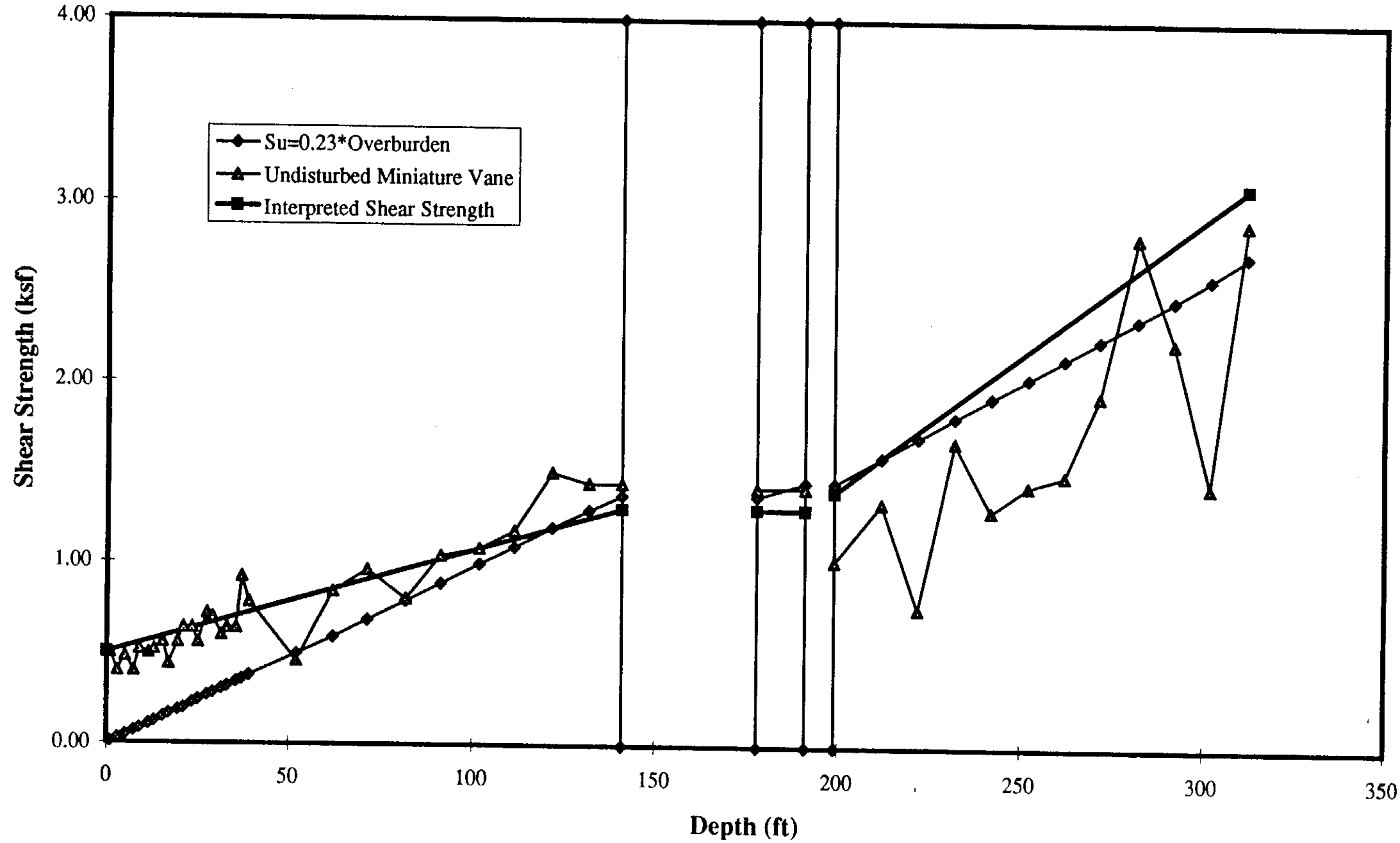


Figure C-3 - Shear Strength Profile - Block Ship Shoal 139- Driven Samples

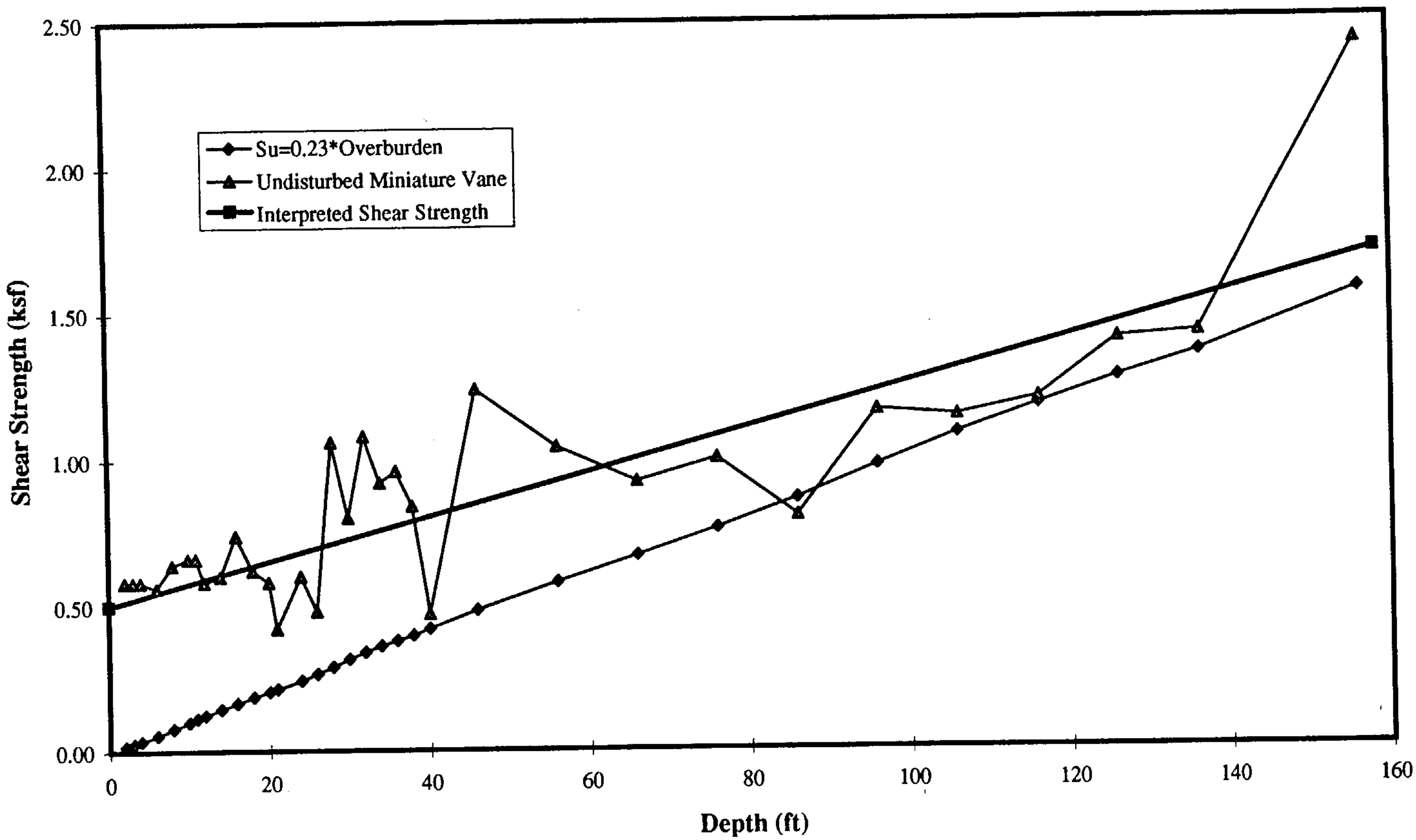


Figure C-4: Shear Strength Profile - Block S. Pelto10 - Driven Samples

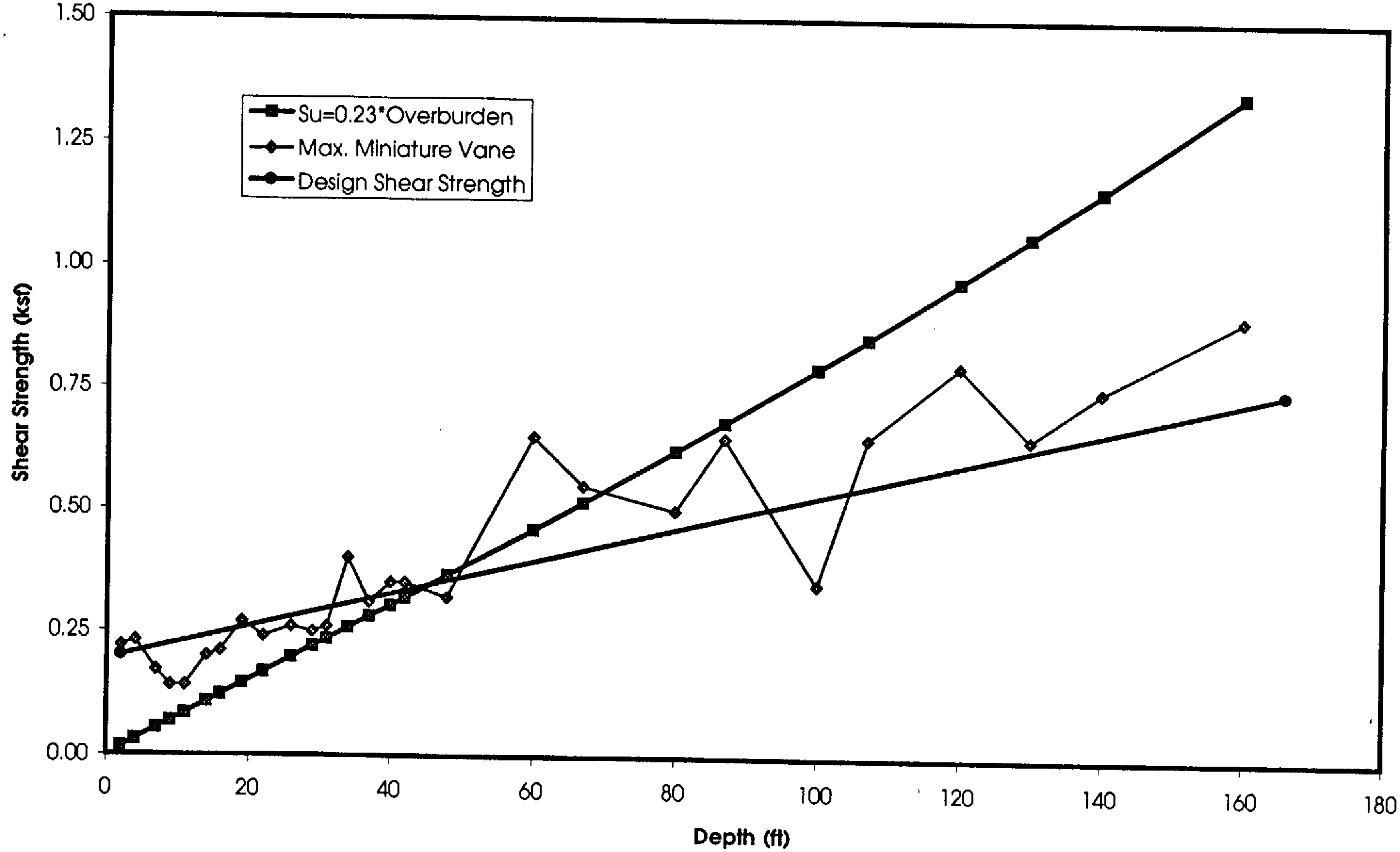


Figure C-5 - Shear Strength Profile - Block Ship Shoal 135 - Driven Samples

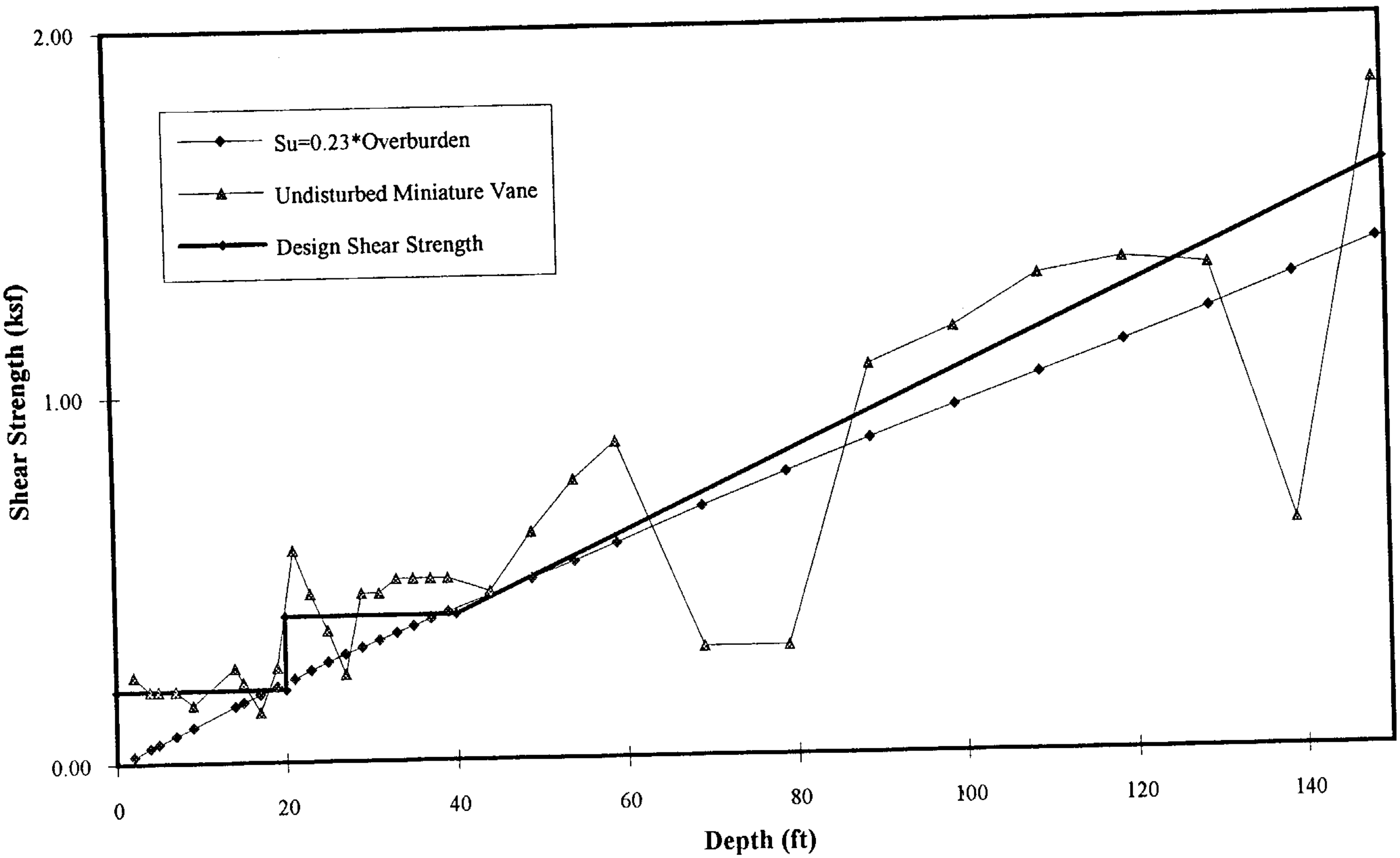
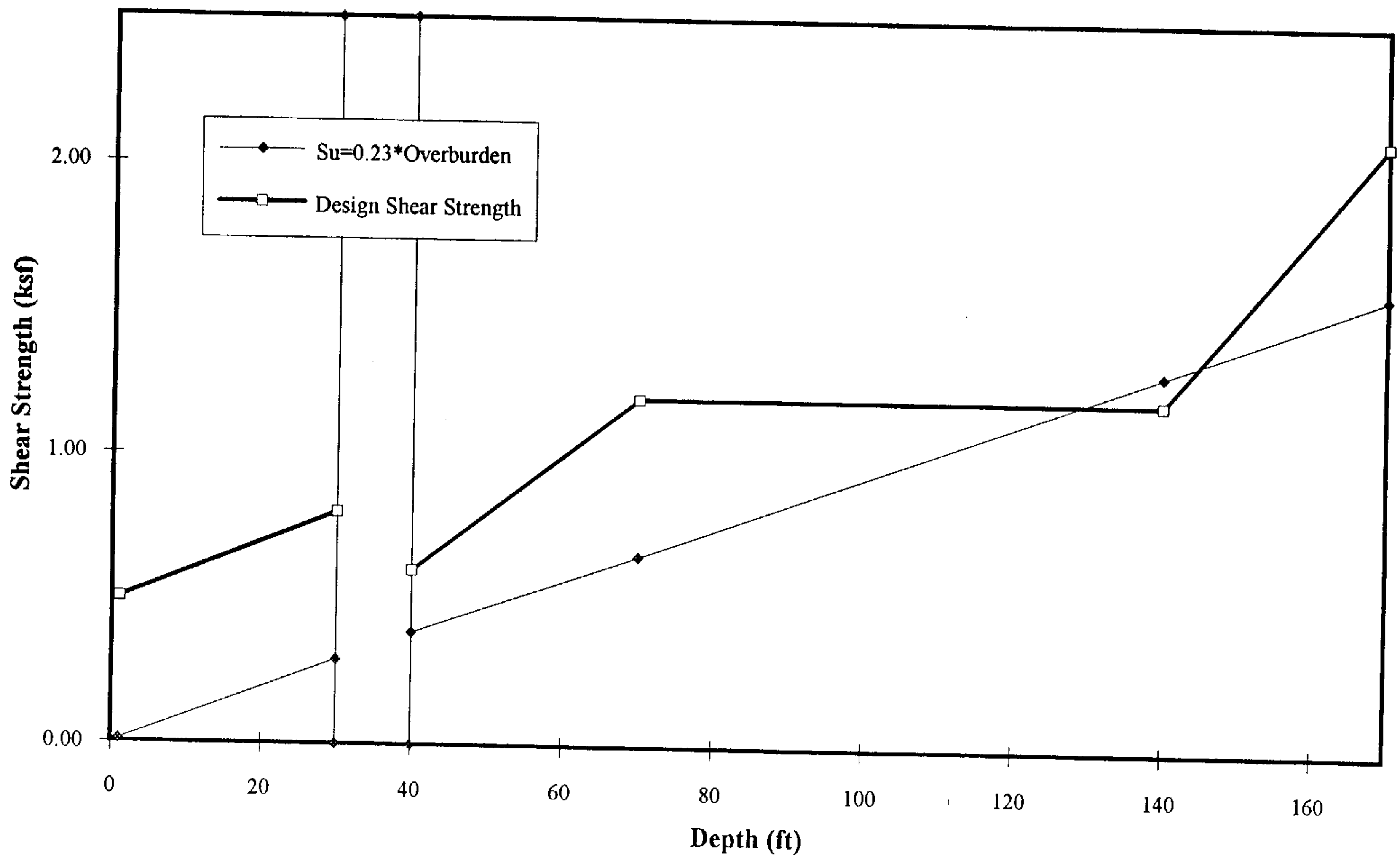
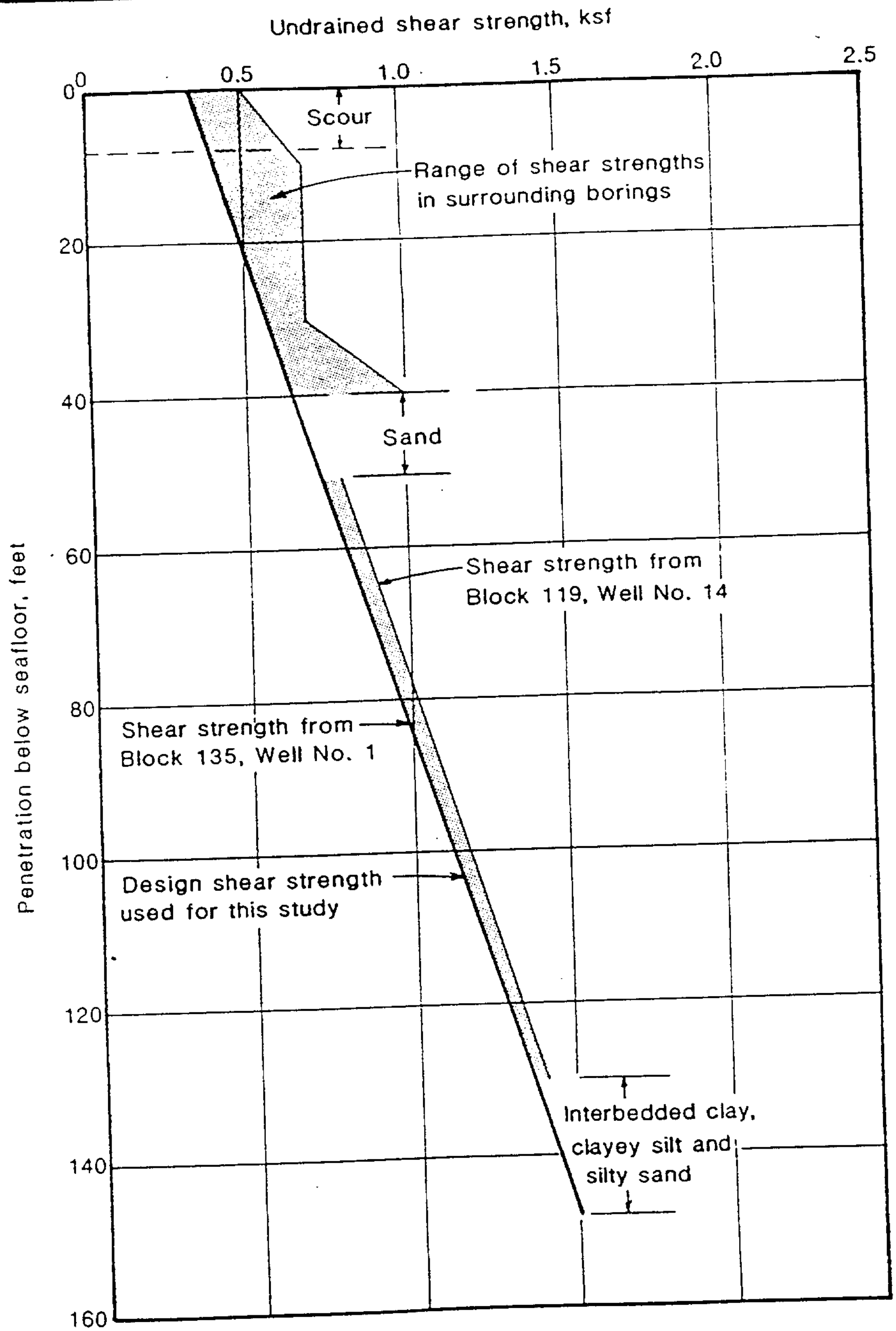


Figure C-6 - Shear Strength Profile - Block Ship Shoal 113 - Driven Samples



SS136



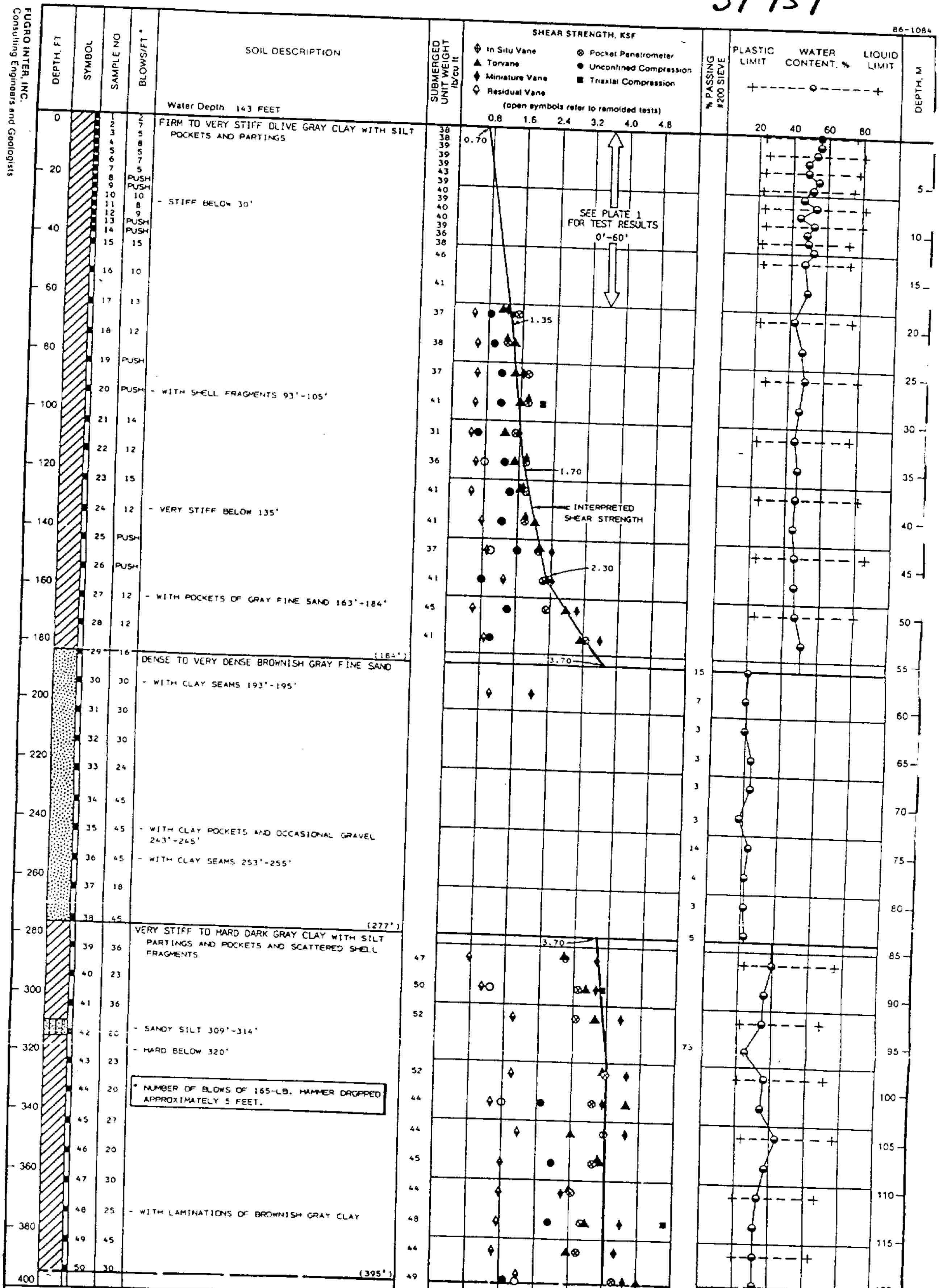
SHEAR STRENGTHS

VICINITY OF WELL NO. 1

BLOCK 136, SHIP SHOAL AREA

Figure C-7

ST 151



Final Penetration: 395 FEET
Field Engineer: G.R. SUNDERLAND

Location: X = 2,347,780 FEET
Y = -16,685 FEET

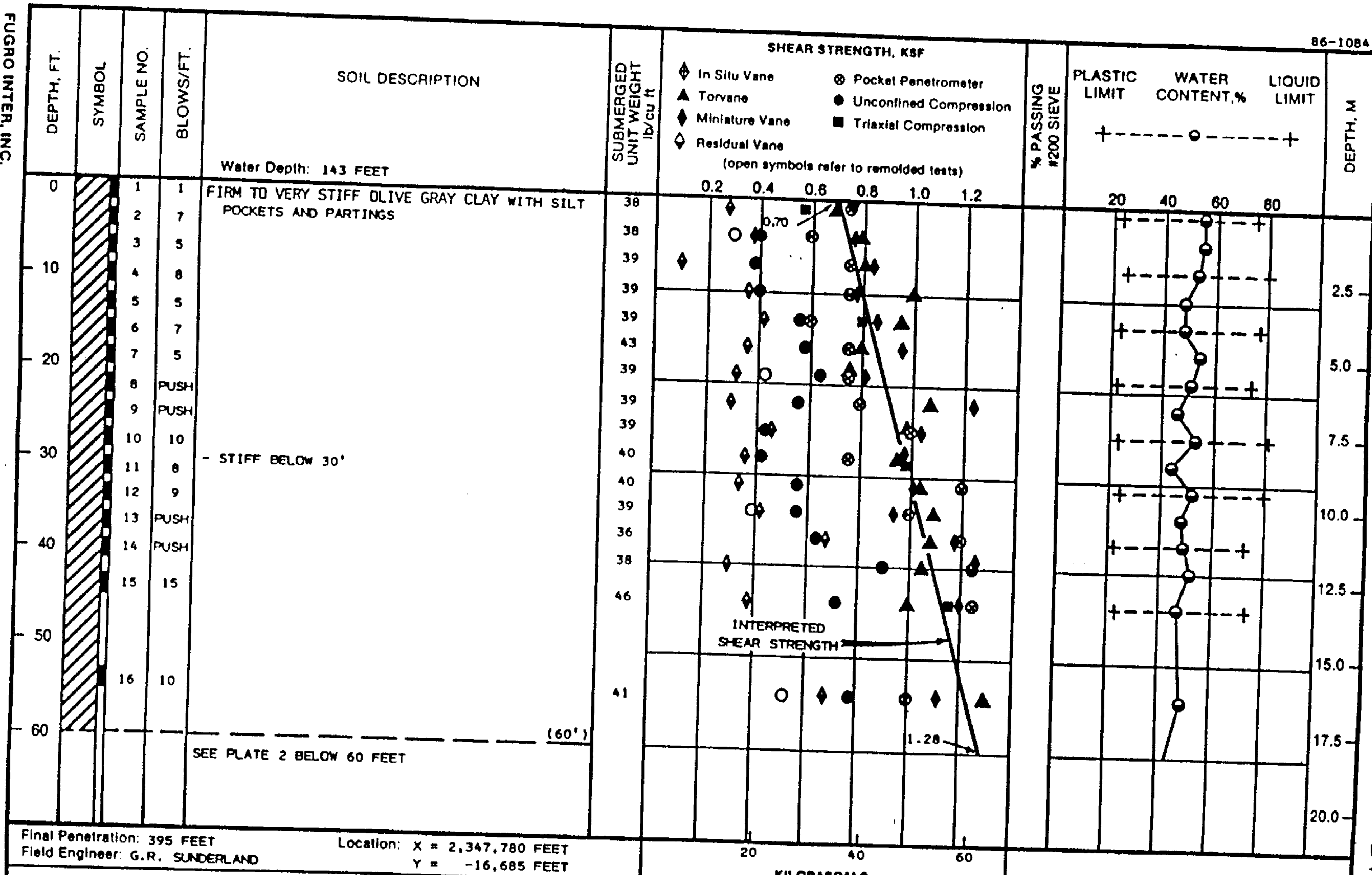
FUGRO INTER, INC.
Consulting Engineers and Geologists

LOG OF BORING AND TEST RESULTS

BLOCK 151, SOUTH TIMBALIER AREA
GULF OF MEXICO

PLATE 2

ST151



Sample No.	Blows/Ft.	Submerged Unit Weight (lb/cu ft)
1	1	38
2	7	38
3	5	39
4	8	39
5	5	39
6	7	43
7	5	39
8	PUSH	39
9	PUSH	39
10	10	40
11	8	40
12	9	39
13	PUSH	36
14	PUSH	38
15	15	46
16	10	41

LOG OF BORING AND TEST RESULTS
0-60 FT.
BLOCK 151, SOUTH TIMBALIER AREA
GULF OF MEXICO

FUGRO INTER, INC.
Consulting Engineers and Geologists

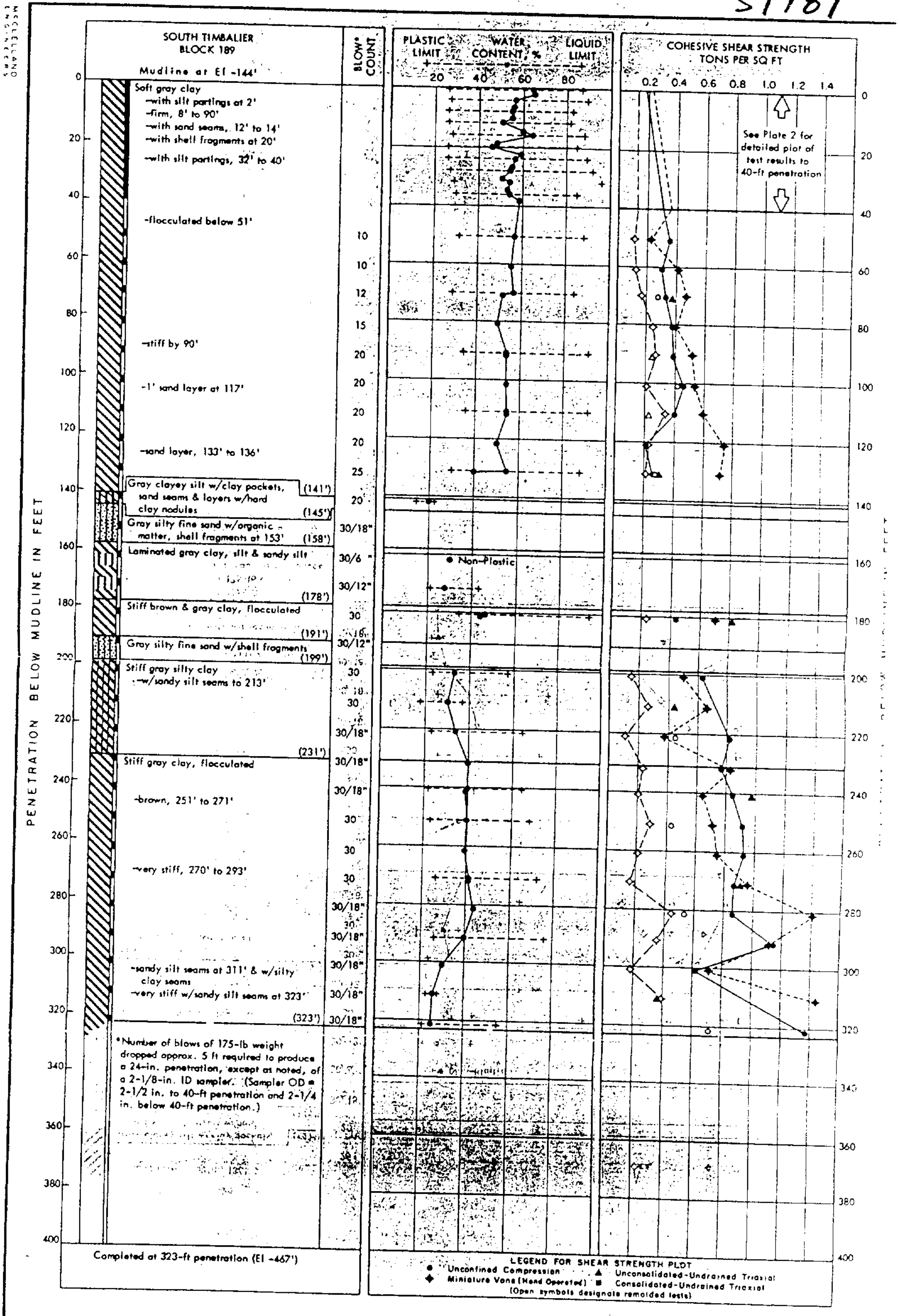
PLATE 1

SAMPLE NO.	DEPTH, FT	BLOWS/FOOT	CLASSIFICATION TESTS: LIQUID LIMIT	CLASSIFICATION TESTS: PLASTIC LIMIT	CLASSIFICATION TESTS: WATER CONTENT, %	CLASSIFICATION TESTS: SUBMERGED UNIT WEIGHT lb/cu.ft.	CLASSIFICATION TESTS: % PASSING #200 SIEVE	SHEAR STRENGTH, Kips/Sq.Ft: PENETROMETER	SHEAR STRENGTH: TORVANE	MINIATURE VANE: UNDISTURBED	MINIATURE VANE: RESIDUAL	UNCONFINED COMPRESSION: UNDISTURBED	UNCONFINED COMPRESSION: REMOLDED	TRIAXIAL COMPRESSION: UNCONSOLIDATED UNDRAINED: UNDISTURBED	TRIAXIAL COMPRESSION: UNCONSOLIDATED UNDRAINED: REMOLDED	TRIAXIAL COMPRESSION: CONSOLIDATED UNDRAINED	Ø, DEGREES
1	1.0	1	76	24	56	38		.75	.70	.76	.28			.57			
2	4.0	7			56	38		.60	.80	.77	.38	.40	.30				
3	7.0	5	81	26	54	39		.75	.82	.84	.10	.38					
4	10.0	8			49	39		.75	1.00	.78	.36	.40					
5	13.0	5	78	24	49	39		.60	.96	.86	.42	.56		.80			
6	16.0	7			55	43		.75	.80	.96	.36	.58					
7	19.0	5	75	23	52	39		.75	.76	.82	.32	.64	.43				
8	22.0	PUSH			47	40		.80	1.08	1.24	.30	.56					
9	25.0	PUSH	82	24	54	39		1.00	1.00	1.04	.46	.44					
10	28.0	10			45	40		.76	.96	.98	.36	.42		.98			
11	31.0	8	80	25	53	40		1.20	1.04	1.02	.34	.56					
12	34.0	9			49	39		1.00	1.10	.94	.42	.56	.39				
13	37.0	PUSH	73	23	50	36		1.20	1.08	1.18	.68	.64					
14	40.0	PUSH			53	38		1.25	1.06	1.26	.30	.90					
15	44.0	15	74	24	48	46		1.25	1.00	1.20	.38	.72		1.16			
16	54.0	10			50	41		1.00	1.30	1.12	.68	.78	.52				
17	64.0	13	76	23	43	37		1.50	1.20	1.24	.46	.84		1.36			
18	74.0	12			48	38		1.25	1.44	1.24	.54	.94					
19	84.0	PUSH	80	27	50	37		1.76	1.44	1.64	.56	1.14					
20	94.0	PUSH			47	41		1.76	1.60	1.76	.54	1.14		2.11			
21	104.0	14	76	23	45	31		1.50	1.24	1.58	.46	.62					
22	114.0	12			47	36		1.75	1.50	1.76	.58	1.26	.78				
23	124.0	15	82	25	46	41		1.76	1.70	1.70	.50	1.40					
24	134.0	12			45	41		1.76	2.00	1.78	.76	1.24					
25	144.0	PUSH	86	24	46	37		2.10	2.14	2.40	.90	1.60	.97				
26	154.0	PUSH			46	41		2.25	2.34	2.42	1.30	.80					
27	164.0	12	81	24	47	45		2.30	2.76	3.02	.60	1.40					
28	174.0	12			51	41		3.25	3.12	3.60	.88	1.02					
29	184.0	16			21		15										
30	194.0	30			20		7			2.02	1.04						
31	204.0	30			20		3										
32	214.0	30			24		3										
33	224.0	24			24		3										
34	234.0	45			18		3										
35	244.0	45			24		14										
36	254.0	45			22		4										
37	264.0	18			22		3										
38	274.0	45			23		5										
39	284.0	36	76	24	40	47		3.00	3.00	3.74	.76						
40	294.0	23			36	50		3.30	3.50	3.74	1.06		1.25	3.89			
41	304.0	36	68	22	35	52		3.26	3.74	4.34	1.80						
42	314.0	20			25		73										
43	324.0	23	71	21	37	52		4.00	4.00	4.50	1.80						
44	334.0	20			35	44		3.70	4.50	3.94	1.32	2.50	1.59				
45	344.0	27	77	24	44	44		4.00	3.20	4.50	1.96						
46	354.0	20			38	45		3.74	3.90	3.86	1.58	2.76					
47	364.0	30	67	20	34	44		3.25	3.25	3.00	1.56						
48	374.0	25			32	48		3.50	3.60	4.40	1.52	2.72		5.42			
49	384.0	45	64	20	32	44		3.40	3.16	4.28	1.42						
50	394.0	30			32	49		4.25	4.80	4.50	2.00	1.70	1.97				

SUMMARY OF TEST RESULTS – Borelog ST151

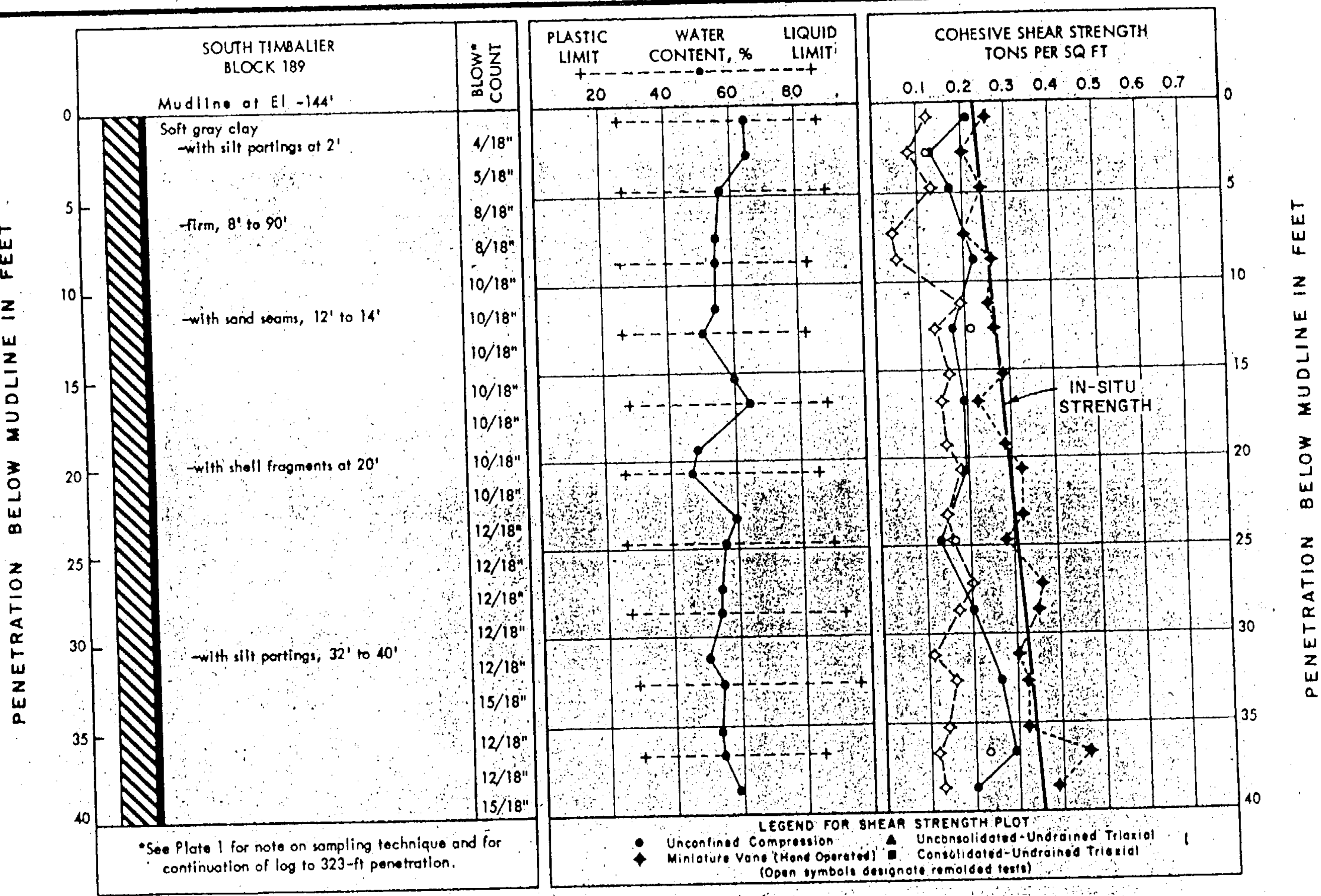
FUGRO INTER, INC.
Consulting Engineers and Geologists

ST189



LOG OF BORING AND TEST RESULTS

PLATE 1

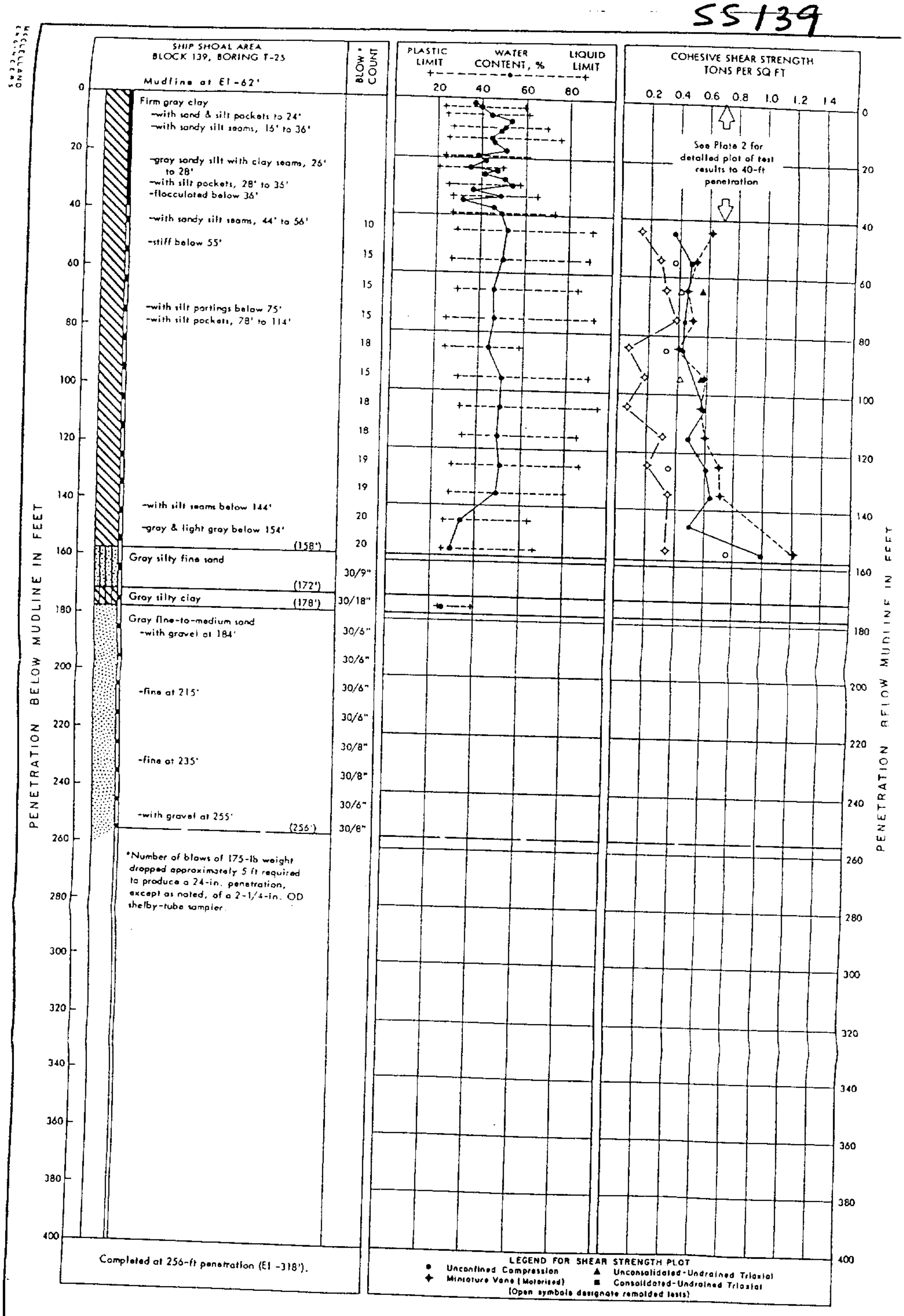


LOG OF BORING AND TEST RESULTS
TO 40-FT PENETRATION

ST189

MCCLELLAND
ENGINEERS

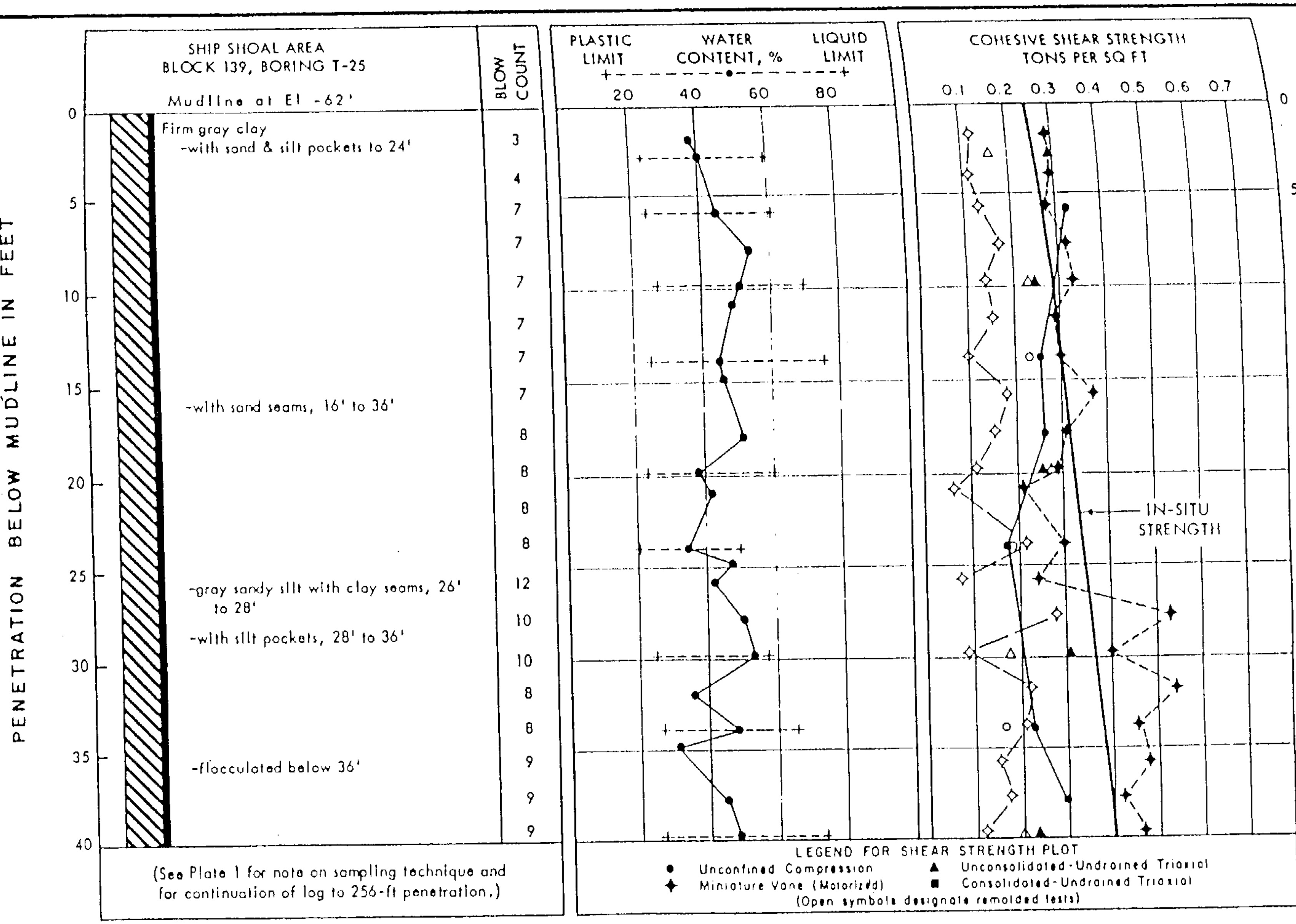
SS139



LOG OF BORING AND TEST RESULTS

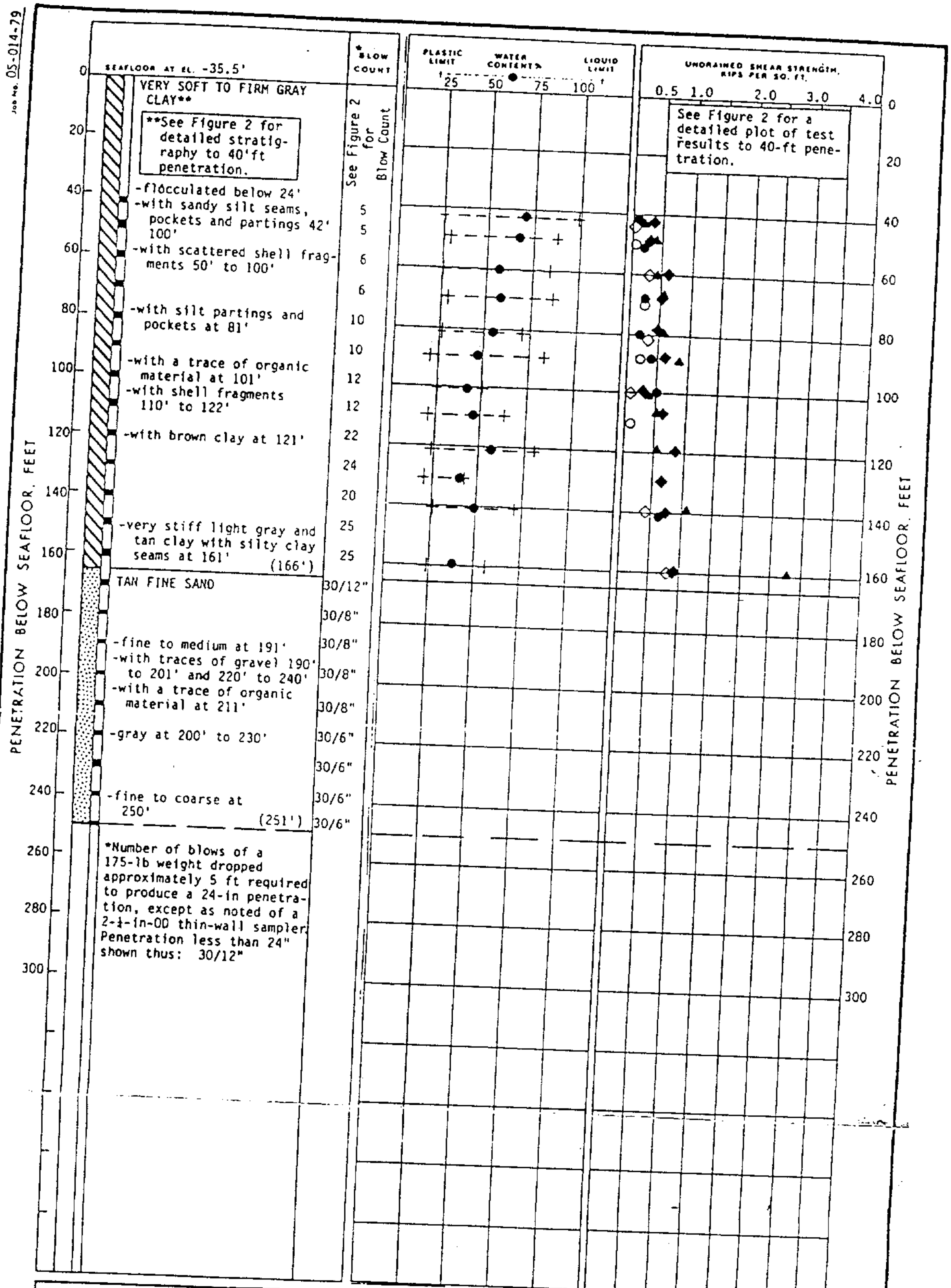
PLATE 1

SS139



LOG OF BORING AND TEST RESULTS
TO 40-FT PENETRATION

McCLELLAND
ENGINEERS



SHEAR STRENGTH LEGEND

- ◆ MINIATURE VANE
- ▲ TORVANE
- ● UNCONFINED COMPRESSION
- ● UNCONSOLIDATED-UNDRAINED TRIAXIAL
- ■ CONSOLIDATED-UNDRAINED TRIAXIAL

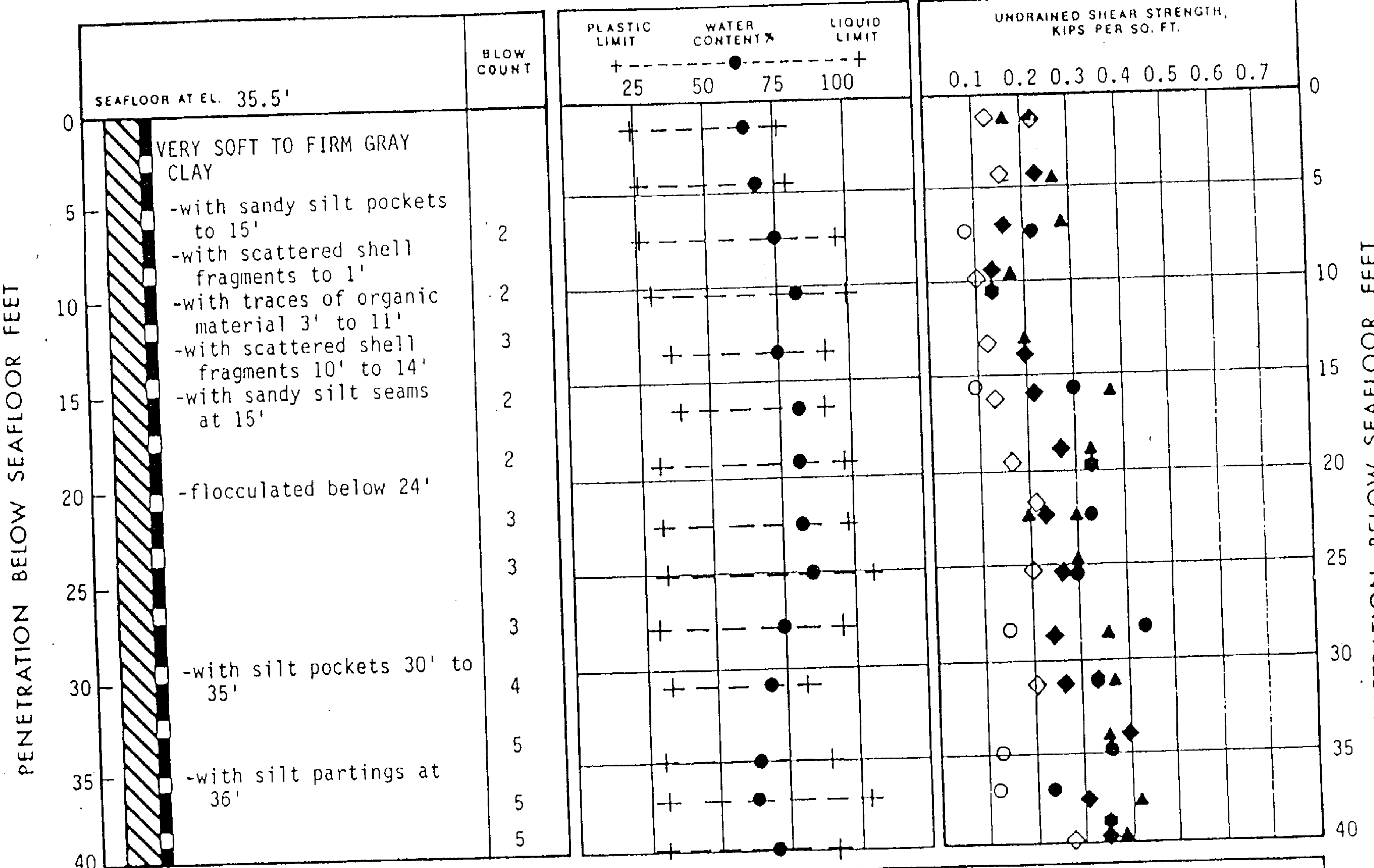
Open Symbols Designate Remolded Tests

LOG OF BORING and TEST RESULTS

MOBIL OIL EXPLORATION & PRODUCING SOUTHEAST, INC.
BLOCK 10, SOUTH PELTO AREA
GULF OF MEXICO

SPelto 10

Job No 05-014-7



SHEAR STRENGTH LEGEND

- ◆ MINIATURE VANE
- ▲ TORVANE
- ● UNCONFINED COMPRESSION
- ⬢ UNCONSOLIDATED–UNDRAINED TRIAXIAL
- ■ CONSOLIDATED–UNDRAINED TRIAXIAL

Open Symbols Designate Remolded Tests

LOG OF BORING and TEST RESULTS

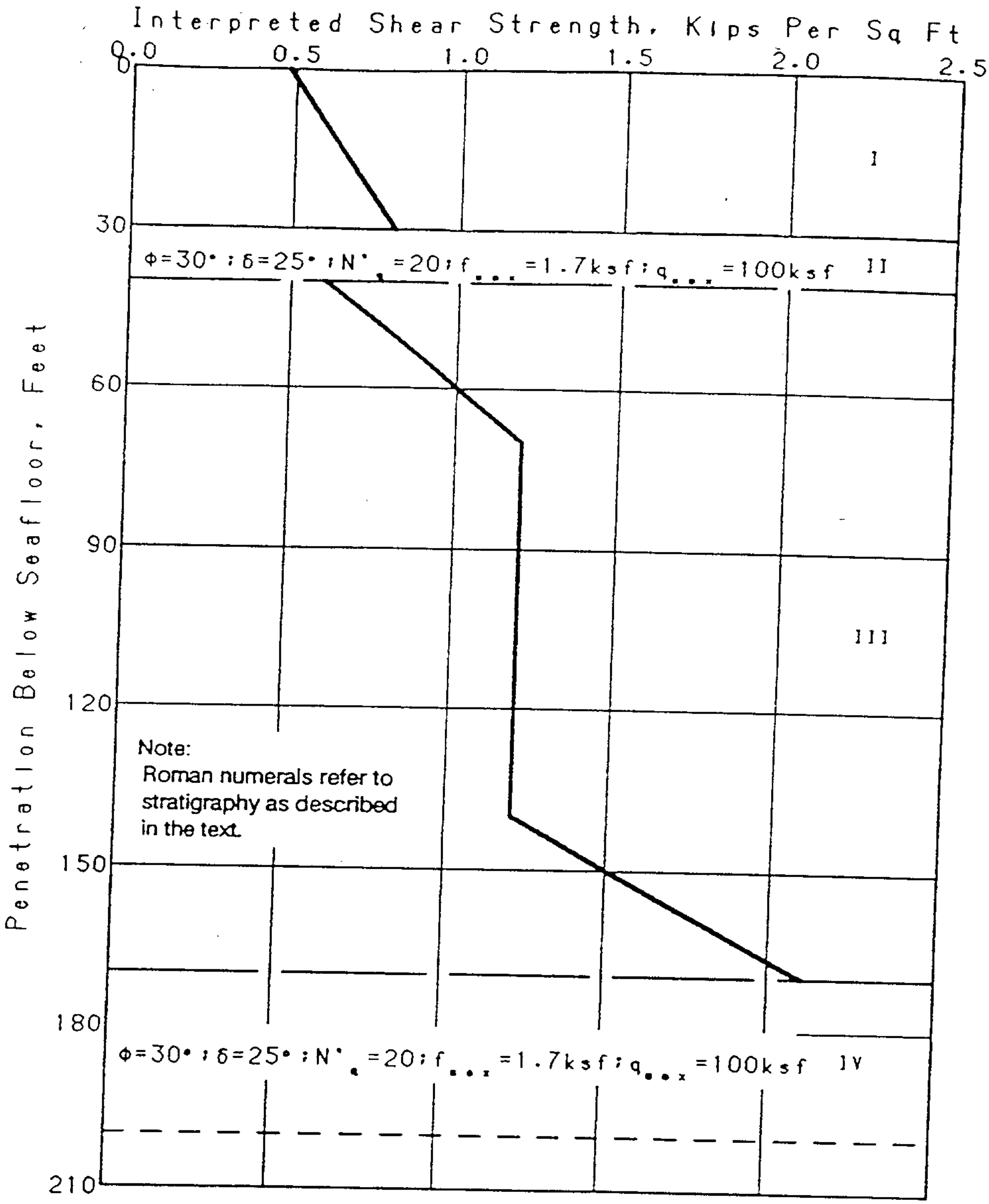
MOBIL OIL EXPLORATION & PRODUCING SOUTHEAST, INC.
BLOCK 10, SOUTH PELTO AREA
GULF OF MEXICO

5 Pelto 10

LAW ENGINEERING TESTING COMPANY

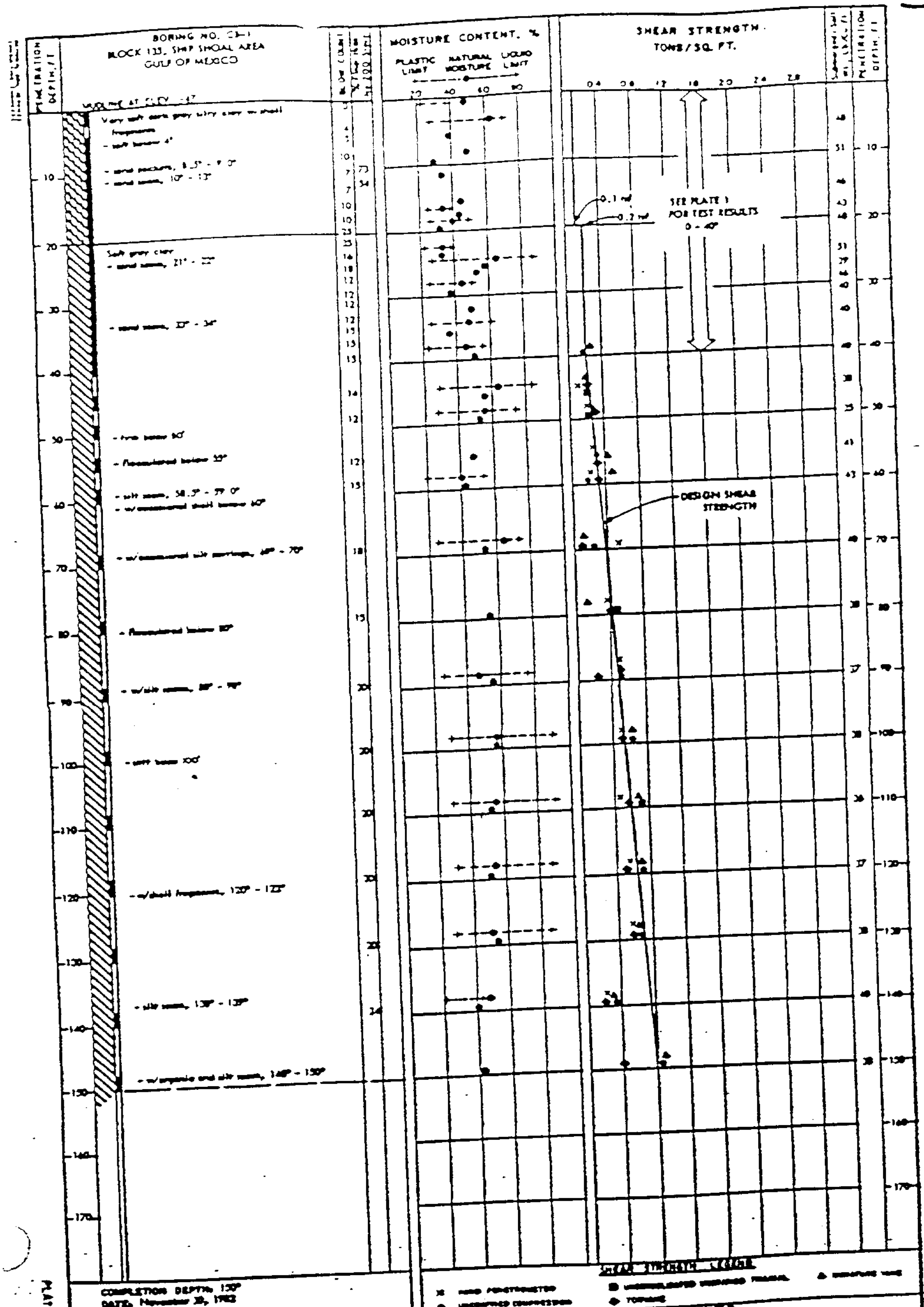
FIGURE 2

SS 113



STRENGTH PARAMETERS
Well No. 48, Block 113
Ship Shoal Area
(Generalized Soil Stratigraphy)

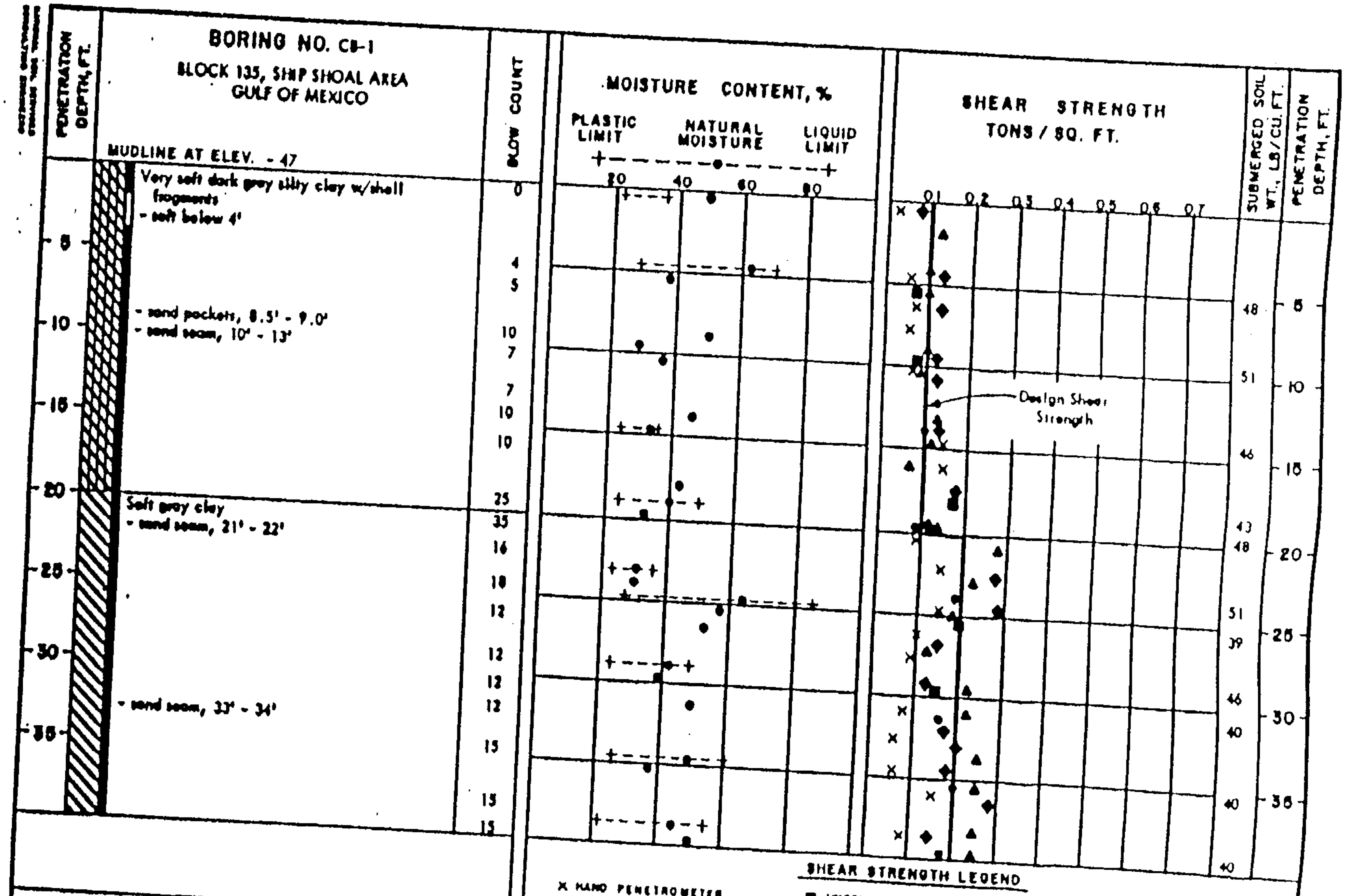
SS135



LOG OF BORING AND SUMMARY OF TEST RESULTS

PLATE 2

SS135



LOG OF BORING AND SUMMARY OF TEST RESULTS TO 40 FOOT PENETRATION

PLATE I