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ABSTRACT

The simplified ultimate strength (SUS) approach to estimating the lateral load ultimate limit states of template-type platforms subjected to hurricane wind and wave forces was used to evaluate six Gulf of Mexico platforms. The results of these evaluations were compared to results obtained using the screening program ULSLEA. Both sets of results were benchmarked against ULS studies performed using the detailed non-linear analysis program USFOS, in order to determine which approach provided the best analytical estimate of platform lateral load capacity. The relative ease of application of both approaches was also discussed.

TABLE OF CONTENTS

1.0 INTRODUCTION

There exist today over 3,800 steel template-type offshore structures in the Gulf of Mexico, sited in water depths ranging from a few fathoms to over 1,000 feet. Many of these structures were designed in the 1950's and 1960's to much lower wave heights and load levels than are prescribed in current codes for the design of new platforms. With these aging structures being called upon to serve beyond their original design life, the need has arisen for accurate means of assessing their ability to survive environmental conditions much more severe than those originally considered.

For the past three years, the Marine Technology and Management Group (MTMG) at U.C. Berkeley has performed a series of research projects, sponsored by regulatory agencies and platform owners, intended to develop and refine analytical screening methodologies that can be used to assess the suitability of aging offshore structures for continued service. This research has concentrated primarily on the development and use of ultimate limit state (ULS) analysis techniques for the purpose of screening steel template-type platforms in the Gulf of Mexico subjected to hurricane wind and wave forces.

Recognizing that very detailed non-linear analyses are impractical for screening large numbers of platforms (due to the effort required), research has concentrated on developing a simple yet accurate screening system for use in platform assessments. The product of this effort has been the "simplified" ULS analysis system known as ULSLEA: Ultimate Limit State Limit Equilibrium Analysis. This system establishes a lateral shear strength capacity profile for a given platform based on the determination of the individual ultimate capacities of three critical components: the deck legs, the jacket bays, and the foundation. Details of the system have been thoroughly documented by Mortazavi and Bea (1996).

As part of the development of ULSLEA, much effort has been made to verify its accuracy and utility. This report represents a continuation of the verification effort. ULS assessments of six Gulf of Mexico platforms have been performed using another screening methodology known as simplified ultimate strength (SUS); this method has been documented by Vannan, et al. (1994). The results of the assessments using SUS have then been compared against those obtained using ULSLEA for the same six platforms. Both sets of results have been benchmarked against the results of detailed studies of the platforms performed using the detailed static pushover analysis program USFOS (Loch, Bea, 1995; Stear, Bea, 1996) in order to make a qualitative assessment as to which simplified approach provides a superior estimate of platform lateral load capacity. Comparison is also made between the relative ease of application of both methods. Suggestions are also made as to areas of screening analysis which are in need of further development and improvement.

1.1 Purpose and Scope

The primary goal of this study is to make a comparison between two platform screening methodologies: ULSLEA and SUS. This comparison involves benchmarking lateral load capacity estimates made with each approach for a set of platforms against estimates made with very detailed static pushover analysis methods. The comparison will also involve making a qualitative assessment as to the ease of application of both methods.

SUS evaluations have been made for the six platforms used as verification cases for the program ULSLEA: Amoco ST161A, PMB Benchmark, Chevron ST151H, Chevron ST151K, Shell SP62A, and Shell SS274A. The results of these analyses are then compared with the results of ULSLEA analyses on these same platforms (the first five having been documented by Mortazavi and Bea, 1996; the ULSLEA analysis of SS274A is appended to this report); both sets of results are benchmarked against the results of detailed evaluations of the platforms using the program USFOS as documented by Loch and Bea (1995) and Stear and Bea (1996).

1.2 Organization of Report

This report is divided into four sections. The first section provides general background on the purpose and scope of the project which has sponsored this report. The second section contains the results of the SUS analyses of the six platforms studied. The third section compares the results of the SUS evaluations to those obtained using ULSLEA; with both results being compared to those obtained using USFOS. Comments are also made as to the ease of application of each approach. The fourth section summarizes the comparisons between the studies, and recommendations for areas of future study are also made. The ULSLEA analysis of Shell SS274A has been appended to this report.

1.3 Acknowledgments

The project to develop and verify ULSLEA was sponsored by ARCO Exploration and Production Technology, Exxon Production Research Company, Mobil Technology Company, Shell Offshore Incorporated, and Unocal Corporation. The parallel project to perform detailed static pushover analyses of six Gulf of Mexico platforms was sponsored by the U.S. Minerals Management Service in cooperation with the California State Lands Commission.

Special appreciation is expressed to Amoco Production Company, Chevron Petroleum Technology Company, Shell Offshore Incorporated, and PMB Engineering for their assistance in providing information on the platforms examined in this study.

2.0 SIMPLIFIED ULTIMATE STRENGTH (SUS)

This section presents the results of the simplified ultimate strength analyses of the six platforms studied: Amoco ST161A, PMB Benchmark, Chevron 151H, Chevron 151K, Shell SP62A, and Shell SS274A. The general procedures used in the analyses are first summarized; subsequent sections describe each platform evaluation. For each platform, a description of the structure is given, along with a short history of the performance of the structure. First member failure and the corresponding lateral load for this event are identified for the principal directions of loading (end-on and broadside), and the results of the analyses are qualitatively compared to historical performance.

2.1 Analysis Approach

The simplified ultimate strength (SUS) approach suggested by Vannan, et al. (1994) entails the performance of linear analyses (i.e. there is no accounting for either material or geometric non-linearities within the solution) of the platform subjected to gradually increasing environmental load conditions. The platform is judged to have reached ultimate strength when one of the following four platform components reaches its ultimate capacity: joints, members, pile steel strength, and pile soil bearing capacity. API RP 2A-LRFD (API, 1993) equations are used to perform checks of the above components, with all load and resistance factors being set to unity. Given the "brittle" behavior (i.e., little or no reserve strength after first member failure) exhibited by these template-type structures, this will result in a reasonable approximation to the platform's ULS.

It was decided for the analyses documented within this report to base the ultimate strength level determination solely on member strength and pile steel strength, due to previous conservatisms identified when evaluating joint performance and pile bearing capacity using API design guidelines (Vannan, et al., 1994; Loch, Bea, 1995; Stear, Bea, 1996). The equation used to evaluate member performance was D.3.2-1 from API RP 2A-LRFD, 1993 (combined axial compression and bending):

$$\frac{f_c}{\varphi_c F_{cn}} + \frac{1}{\varphi_b F_{bn}} \left\{ \left[\frac{C_{my} f_{by}}{\left(1 - \frac{f_c}{\varphi_c F_{ey}}\right)} \right]^2 + \left[\frac{C_{mz} f_{bz}}{\left(1 - \frac{f_c}{\varphi_c F_{ez}}\right)} \right]^2 \right\}^{0.5} \leq 1.0$$

where:

φ_b, φ_c = resistance factors for bending and axial compression, both set to unity

C_{my}, C_{mz} = bending reduction factors, taken as the lesser of $1.0 - 0.4\left(\frac{f_c}{\varphi_c F_e}\right)$ or 0.85

f_c = applied axial compressive stress

F_{cn} = nominal axial compressive strength, as found from:

$F_{cn} = [1.0\text{-}0.25\lambda^2]F_y$ for $\lambda < \sqrt{2}$

$F_{cn} = F_y$ for $\lambda \geq \sqrt{2}$

with $\lambda = \frac{kl}{\pi r}\left(\frac{F_y}{E}\right)^2$

f_{by}, f_{bz} = applied bending stress

F_{bn} = nominal bending strength, as found from:

$F_{bn} = (Z/S)F_y$ for $D/t \leq 1500/F_y$

$F_{bn} = [1.13 - 2.58\,[(F_y D)/(E\,t)]](Z/S)F_y$ for $1500/F_y < D/t \leq 3000/F_y$

$F_{bn} = [0.94 - 0.76\,[(F_y D)/(E\,t)]](Z/S)F_y$ for $3000/F_y$, $D/t \leq 300$

F_{ey}, F_{ez} = Euler buckling strengths, as given by $F_e = F_y/\lambda^2$

It was decided to use an effective length factor of k=0.65 as opposed to the API-recommended value of 0.8, as it has been shown this is more realistic (Vannan, et. al., 1994; Mortazavi, Bea, 1995).

To perform these analyses for this study, it was decided to use the program USFOS (SINTEF, 1994). This program was used previously in the performance of detailed static pushover analyses of the six platforms studied as part of the MMS/CSLC effort documented by Loch and Bea (1995) and Stear and Bea (1996). It was deemed that using this program system would save much time and effort (and minimize possible modeling errors) as the same models used in the performance of the detailed static-pushover analyses could be utilized. To adapt USFOS for "pseudo-linear" analysis, yielding of members was suppressed during the structural analysis. It should be noted that non-linearities due to geometric changes cannot be suppressed in the solution algorithm (they are automatically included in the stiffness formulation); however, as these effects are not expected to be significant for the level of lateral displacement involved, it was judged by the analyst that the error would be minimal. No checks for local failures such as local buckling or fatigue are made, nor are any member imperfections specified. Further description of USFOS and its supporting programs may be found in Loch and Bea (1995) and Stear and Bea (1996).

Only the major structural components were included within the models; the contribution of appurtenances and conductors to the platforms' stiffnesses and strengths were neglected. However, loads induced on the platforms due to these "non-structural" components were taken into account. Also, where F_y is called for in making the member checks, the minimum specified yield strength of the member is increased to account for the true average value of yield strength, and to include strain rate effects. Isometrics and elevations of the structural models may be found in previous reports (Loch, Bea, 1995; Stear, Bea, 1996).

Foundation pile behavior was taken into account through the use of linear spring-to-ground elements. The load-displacement behavior of the spring elements was set equal to the initial slope of the load-displacement curves developed for the static-pushover analyses performed by Loch and Bea (1995) and Stear and Bea (1996) as shown below.

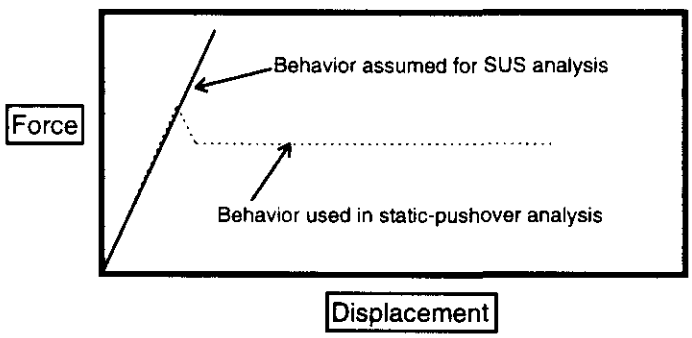

Figure 2.1-1: Foundation Element Force-Displacement Relationships

Loads were applied to the platforms using the same factors and environmental characteristics as those used in the detailed evaluations performed by Loch and Bea (1995) and Stear and Bea (1996); the reader is referred to these reports if more detailed information is desired. The wave heights used in the analyses were those identified as causing collapse from the detailed static pushover analyses; it should be noted that to perform a series of analyses over which the wave height was gradually increased would have been far too time consuming. Only the principal directions of loading (end-on and broadside) were considered.

2.2 Amoco ST161A

2.2.1 Platform Characteristics and History

Amoco ST161A is an eight-leg structure sited in 118 ft of water in the South Timbalier region of the Gulf of Mexico (Figure 2.2.1-1). The platform was designed using a design wave height of 55 ft. The cellar and main decks are located at +36 ft and +47 ft, respectively. The perimeter framing of the jacket is battered to 1:8. The jacket legs are 39 inches in diameter and have no joint cans. The 36 inch-diameter piles were driven to 190 ft, and have been grouted inside the jacket legs. The foundation soils consist primarily of gray clay. The mean yield strengths of the diagonal braces were taken to be 58 ksi, while those of the jacket legs and piles were taken to be 43 ksi.

ST161A has been subjected to two major hurricanes during its service life. Hurricane Carmen passed close by in 1974; damage to the lower decks of the platform suggested that the structure had been subjected to waves up to 58 ft high. A post-hurricane platform condition assessment revealed some damage to the vertical diagonal joints at the top of the uppermost jacket bay. In 1988, the platform was the subject of a comprehensive risk analysis. Consequent risk mitigation measures included removal of the conductors and all equipment from the lower decks. In 1992 Hurricane Andrew passed within a few miles of this platform; damage to the cellar deck and hindcast studies performed following the passage of Andrew suggested waves with heights of 60-64 ft had struck the platform from the ESE. The platform survived with some yielding to the K-joints at the top of the uppermost jacket bay. It was estimated that the absence of the conductors and equipment on the lower decks reduced the total lateral load on the structure by 20%.

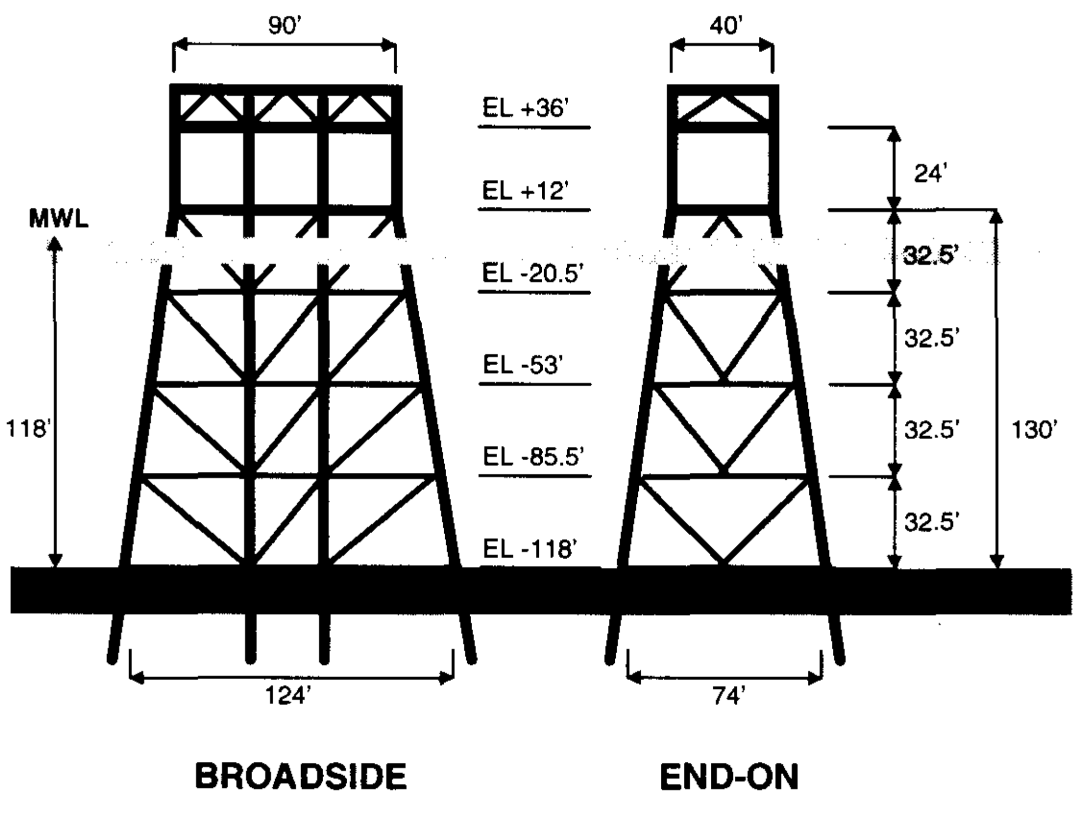

Figure 2.2.1-1 : Amoco ST161A Elevations

2.2.2 Evaluation Results

For the case of end-on wave attack, forces appropriate to a 64 ft wave together with wind and current were used. This corresponds to a total lateral load of 3,486 kips. The first member to reach the API failure criteria under this loading was diagonal brace A3-A4 in the top jacket bay (see Figure 2.2.2-1); the failure criteria was reached at a load factor of 0.74, which represents a lateral load of 2,580 kips.

For the case of broadside wave attack, forces appropriate to a 64 ft wave together with wind and current were used; this corresponds to a total lateral load of 3,809 kips. The first member to reach the failure criteria for this loading was diagonal brace center-B1 in the top jacket bay. This brace reached the criteria at a load factor of 0.82, which corresponds to a lateral load of 3,123 kips.

These results indicate the platform would not pass a screening evaluation for waves on the order of 64 ft-high, being only to withstand 74% of the associated load for end-on wave attack and 82% of the associated load for broadside wave attack. Given that the platform was subjected to broadside waves on the order of 60-64 ft high during Hurricane Andrew and survived, it appears this screening assessment is very conservative.

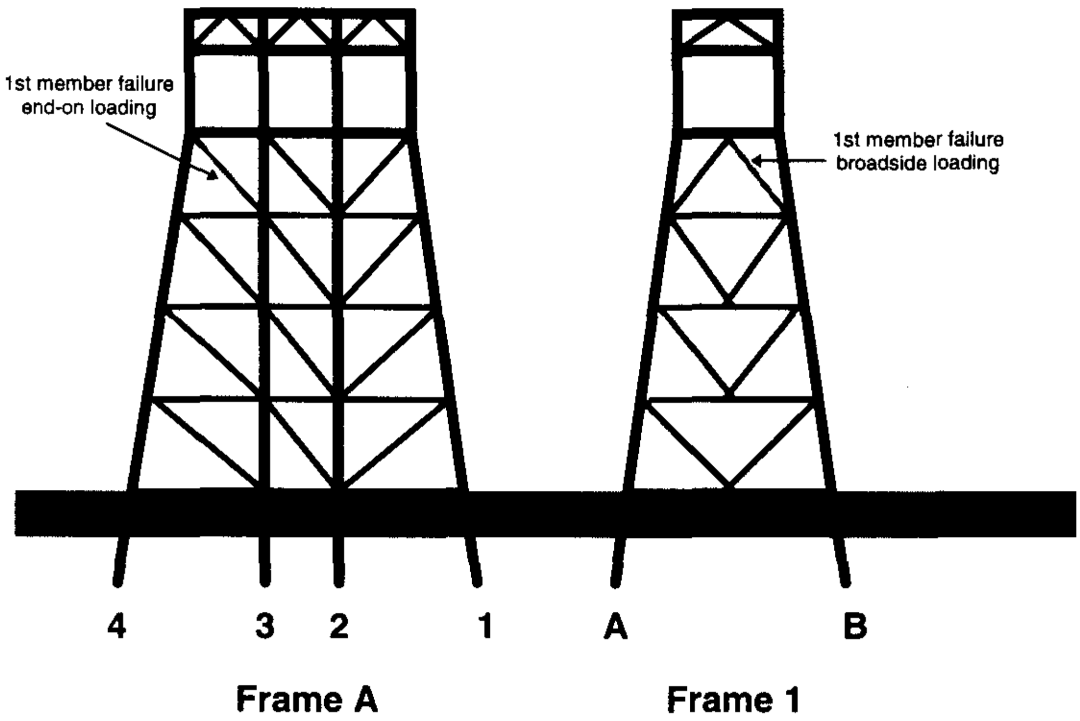

Figure 2.2.2-1: First Member Failures

2.3 PMB Benchmark

2.3.1 Platform Characteristics and History

The PMB Benchmark platform is a self-contained four-pile drilling and production structure (Figure 2.3.1-1). It was installed in the Gulf of Mexico's Ship Shoal region in a water depth of 157 ft in 1971. The platform has four decks at elevations +33 ft, +43 ft, +56 ft, and +71 ft. The jacket legs are battered to 1:11 in the two principal directions and have joint cans. The leg-pile annulus is ungrouted; the piles are attached to the jacket with welded shimmed-connections at the top of the jacket. The piles reach a penetration of 355 ft in soft to stiff gray clay. A mean yield stress of 43 ksi was used for all structural elements of this platform.

This platform was located close to the path of Hurricane Andrew, which passed through the region in 1992. Hindcast studies for the site revealed an estimated maximum wave height of approximately 60 ft. The platform survived the storm without significant damage.

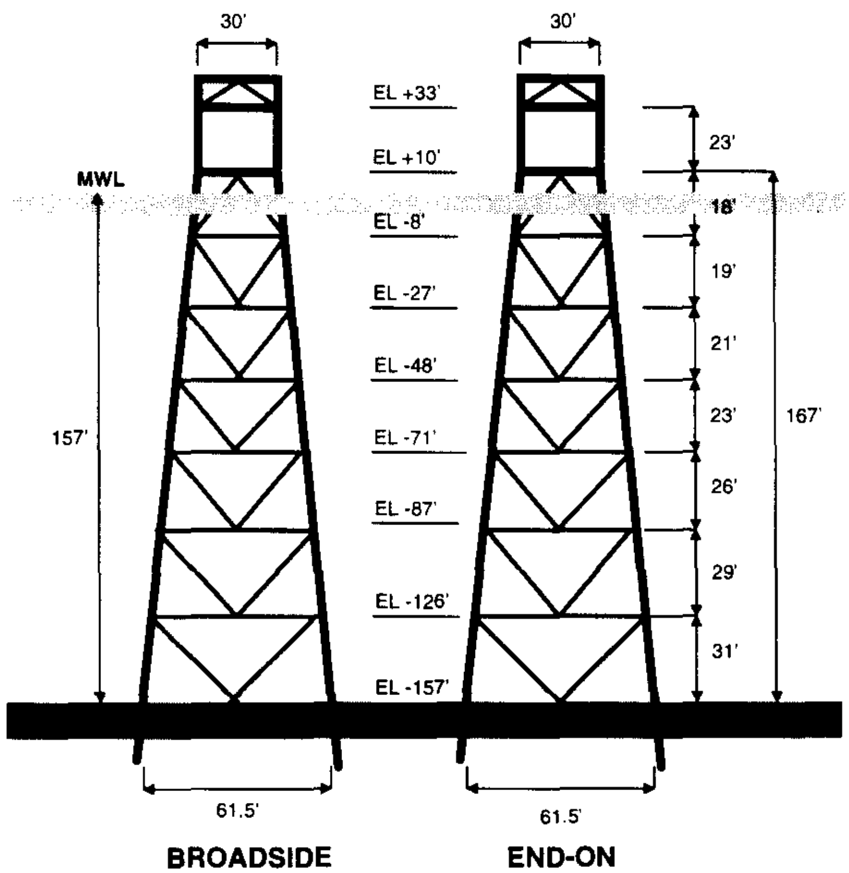

Figure 2.3.1-1: PMB Benchmark Platform Elevations

2.3.2 Evaluation Results

The PMB Benchmark platform was analyzed for the case of end-on wave attack only, due to the similar nature of its E-W and N-S framing. For the case of end-on wave attack, forces appropriate to a 67 ft wave together with wind and current were used. This corresponds to a base shear of 2,656 kips. The first member to reach the failure criteria under this loading was the compression brace in the fifth jacket bay of the east frame of the platform (see Figure 2.3.2-1); the failure criteria was reached at a load factor of 1.25, which represents a base shear of 3,320 kips.

The screening assessment indicates that the PMB Benchmark platform would survive waves on the order of 67 ft high; hence, survival during Andrew is indicated as very likely.

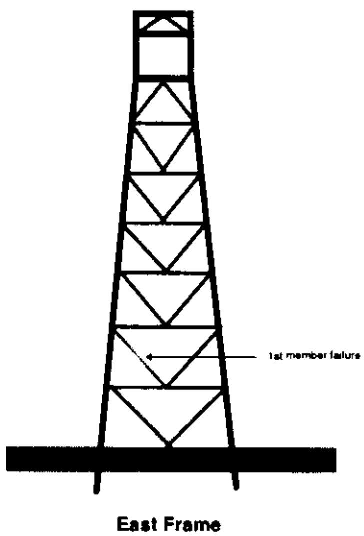

Figure 2.3.2-1: First Member Failure

2.4 <u>Chevron ST151H</u>

2.4.1 Platform Characteristics and History

Chevron ST151H was an eight-pile drilling and production platform located in the Gulf of Mexico's South Timbalier region in 137 feet of water (Figure 2.4.1-1). This region was subjected to 100-year wave loads during Hurricane Andrew in 1992 (Vannan et al., 1994). The platform was designed and installed in 1964. Cellar and main deck elevations are at +35 ft and +46 ft respectively. The broadside frames are battered to 1:12. The 30 inch-diameter piles extend approximately 180 ft below the mudline through firm to very stiff clay. A dense sand layer lies directly beneath the piles ends. The 30 inch-diameter deck legs are connected to the tops of the piles. The 33 inch-diameter legs are ungrouted but have thickened joint sections. The jacket bracing and horizontal framing are made of nominal 36 ksi steel with an average yield strength of 43 ksi.

Chevron ST151H was located close to the path of Hurricane Andrew and collapsed during the storm. Hindcast data indicates that the platform was subjected to end-on attack by 60 ft-high waves; it is believed ST151H was destroyed due to jacket failure in the end-on direction (Vannan, et al., 1994).

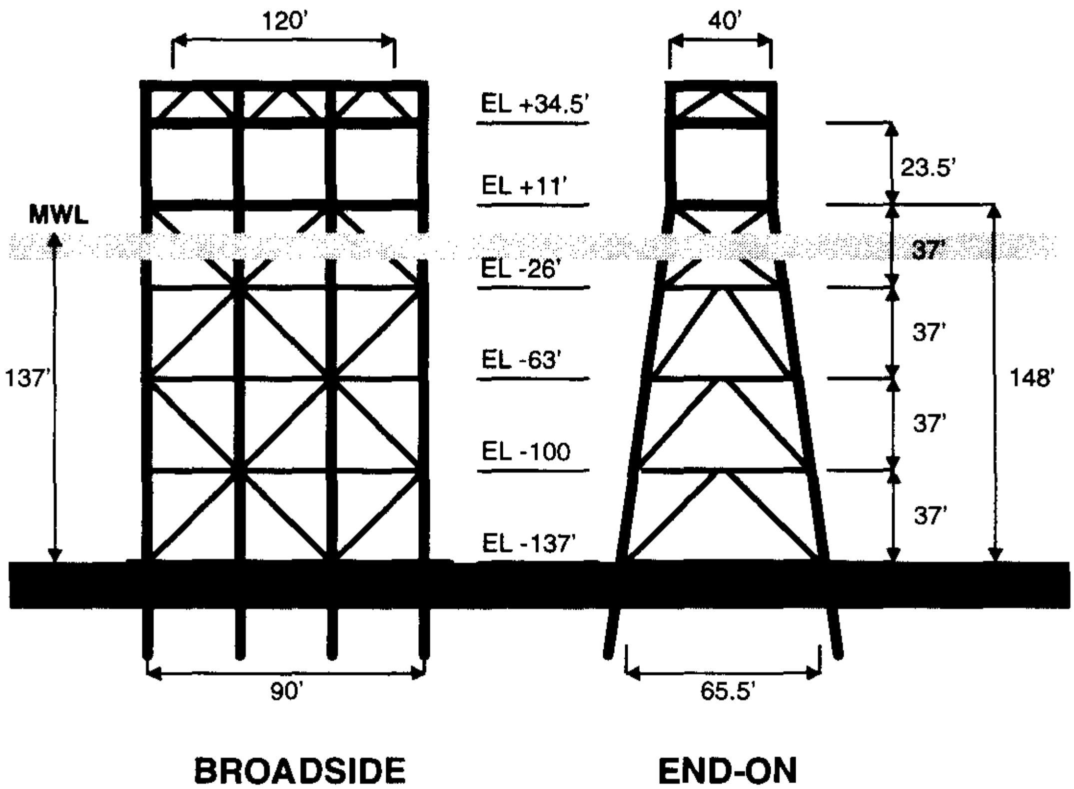

Figure 2.4.1-1: Chevron ST151H Elevations

2.4.2 Evaluation Results

For the case of end-on wave attack, forces appropriate to a 56 ft wave together with wind and current were used. This corresponds to a base shear of 4,333 kips. The first member to reach the failure criteria under this loading was diagonal brace C2-D2 in the bottom jacket bay (see Figure 2.4.2-1); the failure criteria was reached at a load factor of 0.53, which represents a base shear of 2,296 kips.

For the case of broadside wave attack, forces appropriate to a 60 ft wave together with wind and current were used; this corresponds to a base shear of 5,068 kips.

The first member to reach the failure criteria for this loading was diagonal brace center-B2 in the second jacket bay. This brace reached the criteria at a load factor of 0.85, which corresponds to a base shear of 4,308 kips.

The screening assessment indicates ST151H could not survive waves on the order of 55-60 ft high; hence, collapse during Andrew would certainly be expected.

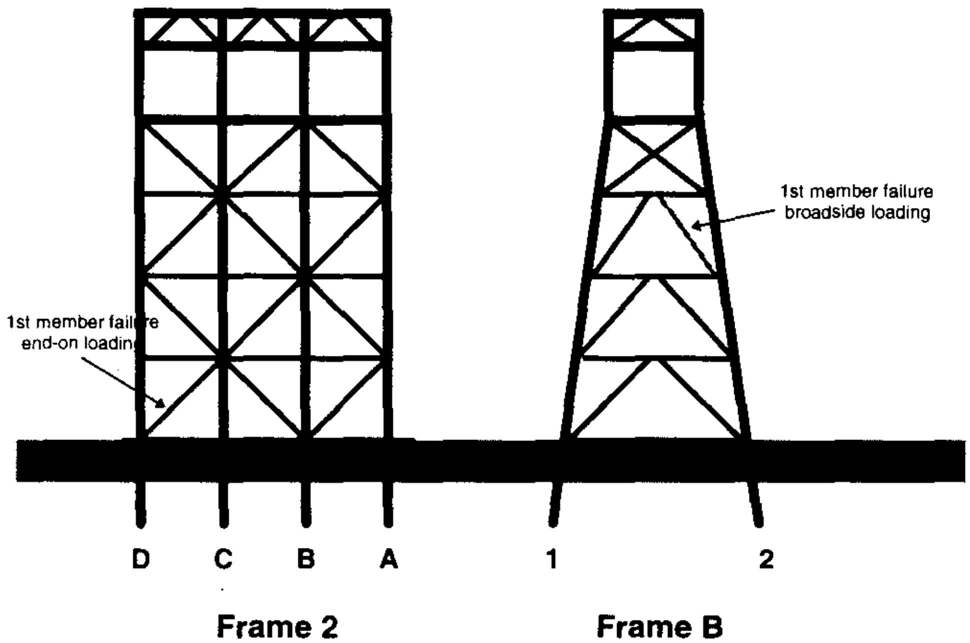

Figure 2.4.2-1: First Member Failure

2.5 <u>Chevron ST151K</u>

2.5.1 Platform Characteristics and History

Chevron ST151K is an eight-leg drilling and production platform located in the Gulf of Mexico's South Timbalier region (Figure 2.5.1-1). This platform was bridge-connected to ST151H, having been designed and installed about the same time. ST151K is similar in geometry to ST151K except that it is battered 1:10 in both broadside and end-on framing. The same wave and wind conditions and force coefficients were used for both platforms. However, ST151K is subjected to a much larger lateral load for the same storm conditions due to its additional conductors.

ST151K was subjected to significant environmental loading during Hurricane Andrew, but unlike ST151H did not collapse. Hindcast data indicates that the platform was subjected to end-on attack by 60 ft-high waves (the same waves which destroyed the sister platform).

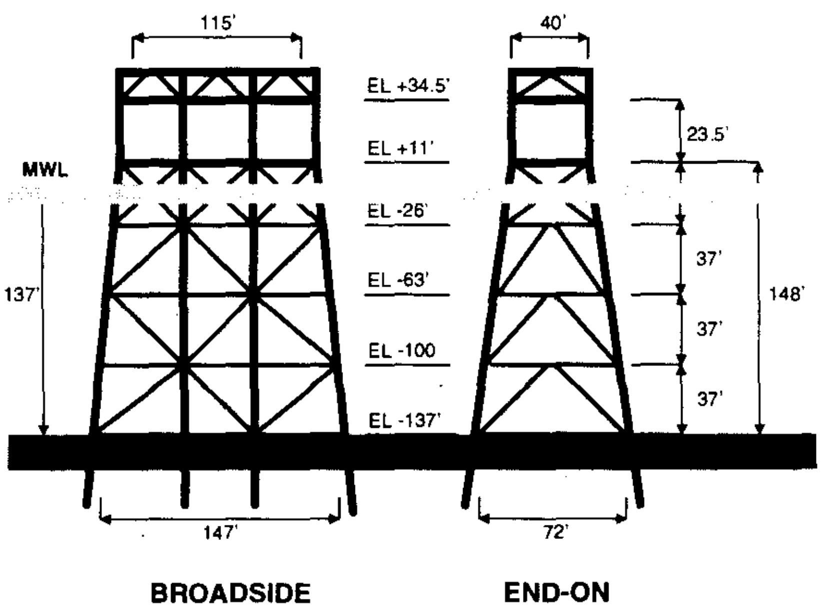

Figure 2.5.1-1 Chevron ST151K Elevations

2.5.2 Evaluation Results

For the case of end-on wave attack, forces appropriate to a 56 ft wave together with wind and current were used. This corresponds to a base shear of 5,226 kips. The first member to reach the failure criteria under this loading was diagonal brace C2-D2 in the bottom jacket bay (see Figure 2.5.2-1); the failure criteria was reached at a load factor of 0.75, which represents a base shear of 3,919 kips.

For the case of broadside wave attack, forces appropriate to a 60 ft wave together with wind and current were used; this corresponds to a base shear of 5,936 kips. The first member to reach the failure criteria for this loading was diagonal brace center-D2 in the second jacket bay. This brace reached the criteria at a load factor of 0.75, which corresponds to a base shear of 4,452 kips.

The screening assessment indicates ST151K would not survive waves on the order of 55-60 ft-high, being only to withstand 75% of the associated load for end-on wave attack and 75% of the associated load for broadside wave attack. Given that the platform was subjected to end-on waves on the order of 60 ft-high during Hurricane Andrew and survived, it appears this screening assessment is very conservative.

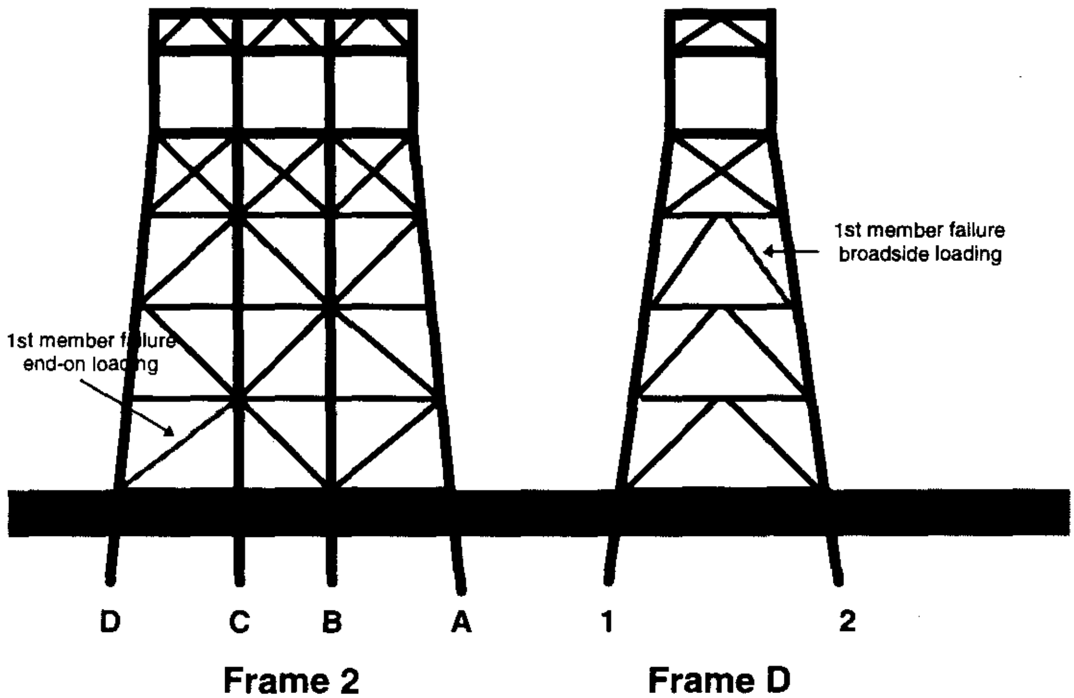

Figure 2.5.2-1: First Member Failure

2.6 Shell SP62A

2.6.1 Platform Characteristics and History

Shell SP62A is an eight-leg self-contained drilling and production platform located in 340 ft of water in the South Pass area of the Gulf of Mexico (Figure 2.6.1-1). The bottom of the lower decks has an elevation of +45 ft. The jacket framing consists of diagonal braces with diameters ranging from 24 to 30 inches and thicknesses from 0.625 to 1.25 inch. This platform supports eighteen 24 inch-diameter conductors. The main piles are driven through the jacket legs and shim-connected to the top of the legs. The jacket leg-pile annulus is not grouted. The platform has 8 additional skirt piles which are grouted in skirt pile legs that are within the framing of the bottom jacket bay. All of the piles penetrate to a depth of 180 ft below mudline. The foundation soils consist predominantly of sand.

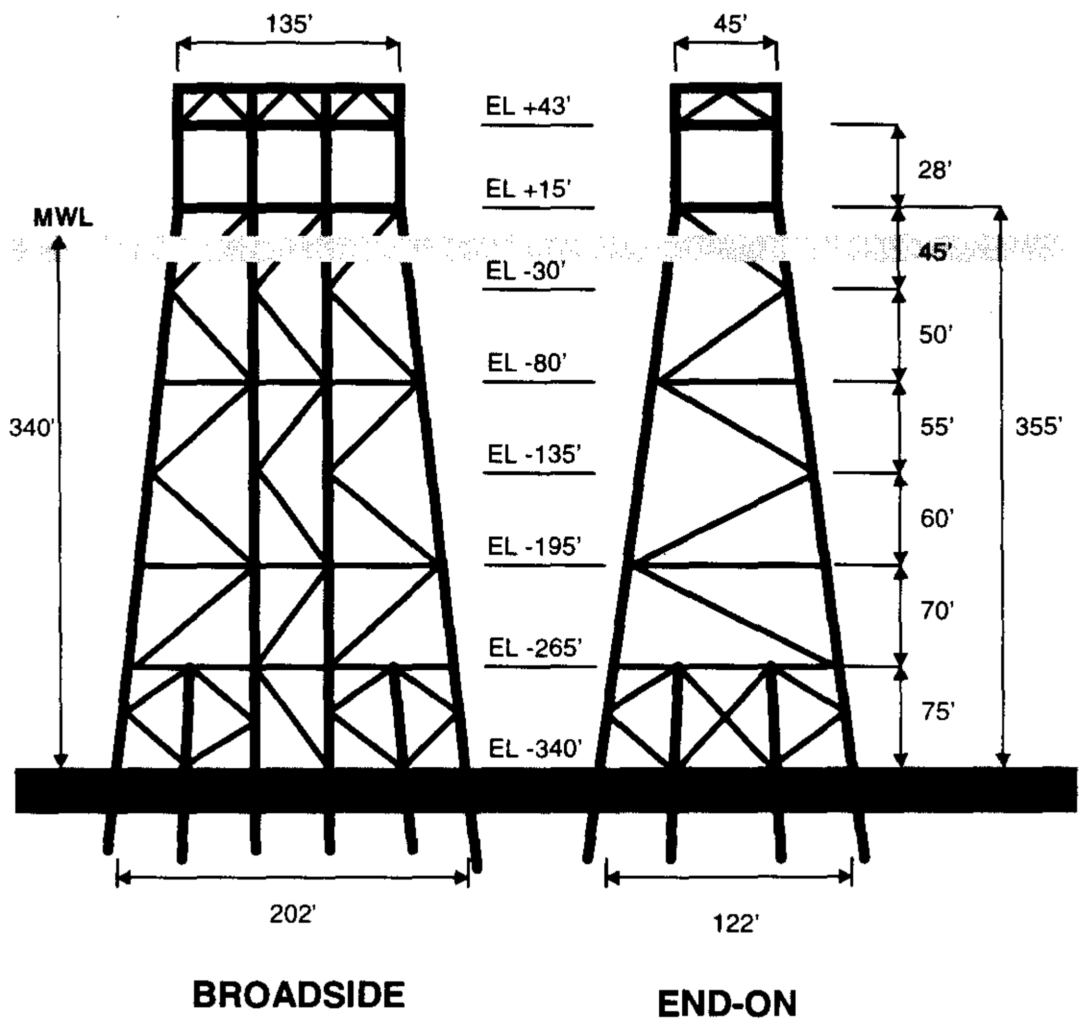

Figure 2.6.1-1: Shell SP62A Elevations

SP62A was subjected to extreme wave loads during Hurricane Camille in 1969, and survived without damage. The maximum wave heights during the storm were estimated to be approximately 75 ft, approaching the platform from the south.

2.6.2 Evaluation Results

For the case of end-on wave attack, forces appropriate to a 80 ft wave together with wind and current were used. This corresponds to a base shear of 7,085 kips. The first member to reach the failure criteria under this loading was diagonal brace A3-A4 in the fourth jacket bay (see Figure 2.6.2-1); the failure criteria was reached at a load factor of 0.85, which represents a base shear of 6,022 kips.

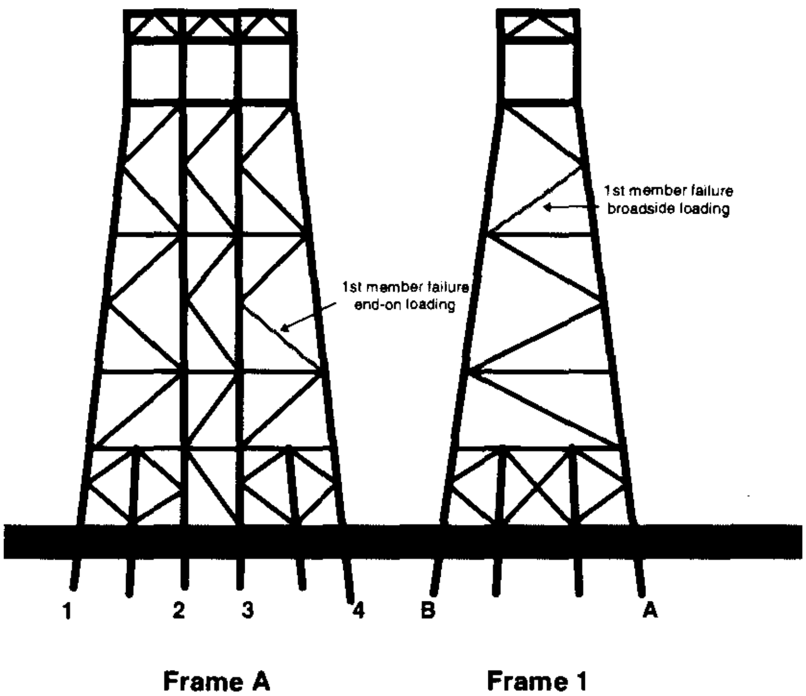

Figure 2.6.2-1: First Member Failure

For the case of broadside wave attack, forces appropriate to a 84 ft wave together with wind and current were used; this corresponds to a base shear of 8,567 kips. The first member to reach the failure criteria for this loading was diagonal brace A1-B1 in the second jacket bay (see Figure 2.6.2-2). This brace reached the criteria at a load factor of 1.0.

Given that the screening analysis indicates the platform can withstand 85% of the load associated with end-on 80 ft waves, it seems likely the platform would be approved for service subjected to 75 ft waves. However, further analysis would need to be performed in order to refine this estimate.

2.7 Shell SS274A

2.7.1 Platform Characteristics and History

Shell SS274A is an eight-leg self-contained drilling and production platform located in 213 ft of water in the Ship Shoal area of the Gulf of Mexico (Figure 2.7.1-1). The bottom of the lower decks has an elevation of +43 ft. The jacket framing consists of diagonal braces with diameters ranging from 16 to 26 inches with thicknesses of 0.5 inch. This platform supports twelve 24 inch-diameter conductors. The main piles are driven through the jacket legs and shim-connected to the top of the legs. The jacket leg-pile annulus is not grouted. All of the piles penetrate to a depth of 250 ft below the mudline. The foundation soils consist predominantly of stiff clay.

SS274A was subjected to extreme wave loads during Hurricane Hilda in 1964. The maximum wave heights during the storm were estimated to be approximately 56 ft. The platform survived the storm, but sustained damage to portions of the jacket legs and several diagonal braces. The damage to the jacket legs was attributed to local buckling, while the damage to the braces is believed to be fatigue-related.

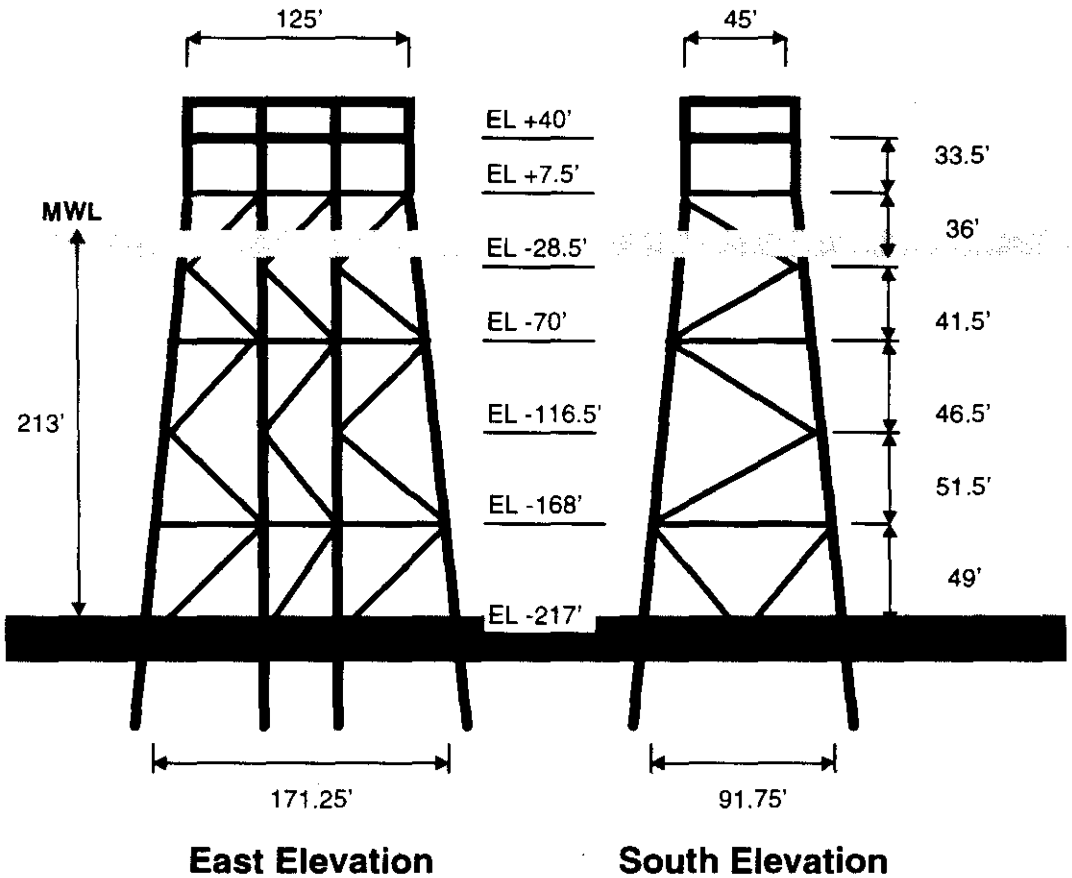

Figure 2.7.1-1: Shell SS274A Elevations

2.7.2 Analysis Results

For the case of end-on wave attack, forces appropriate to a 67 ft wave together with wind and current were used. This corresponds to a base shear of 4,775 kips. The first member to reach the failure criteria under this loading was diagonal brace A3-A4 in the fourth jacket bay (see Figure 2.7.2-1); the failure criteria was reached at a load factor of 0.75, which represents a base shear of 3,581 kips.

For the case of broadside wave attack, forces appropriate to a 67 ft wave together with wind and current were used; this corresponds to a base shear of 5,300 kips. The first member to reach the failure criteria for this loading was diagonal brace A1-B1 in the second jacket bay. This brace reached the criteria at a load factor of 0.8, corresponding to a base shear of 4,240 kips.

SS274A would not pass a screening evaluation for waves on the order of 67 ft-high, being only to withstand 75% of the associated load for end-on wave attack and 80% of the associated load for broadside wave attack. However, it seems that the screening would indicate survival during Hurricane Hilda as being likely. It should be noted, however, that this screening assessment will not capture local failures of the type actually observed.

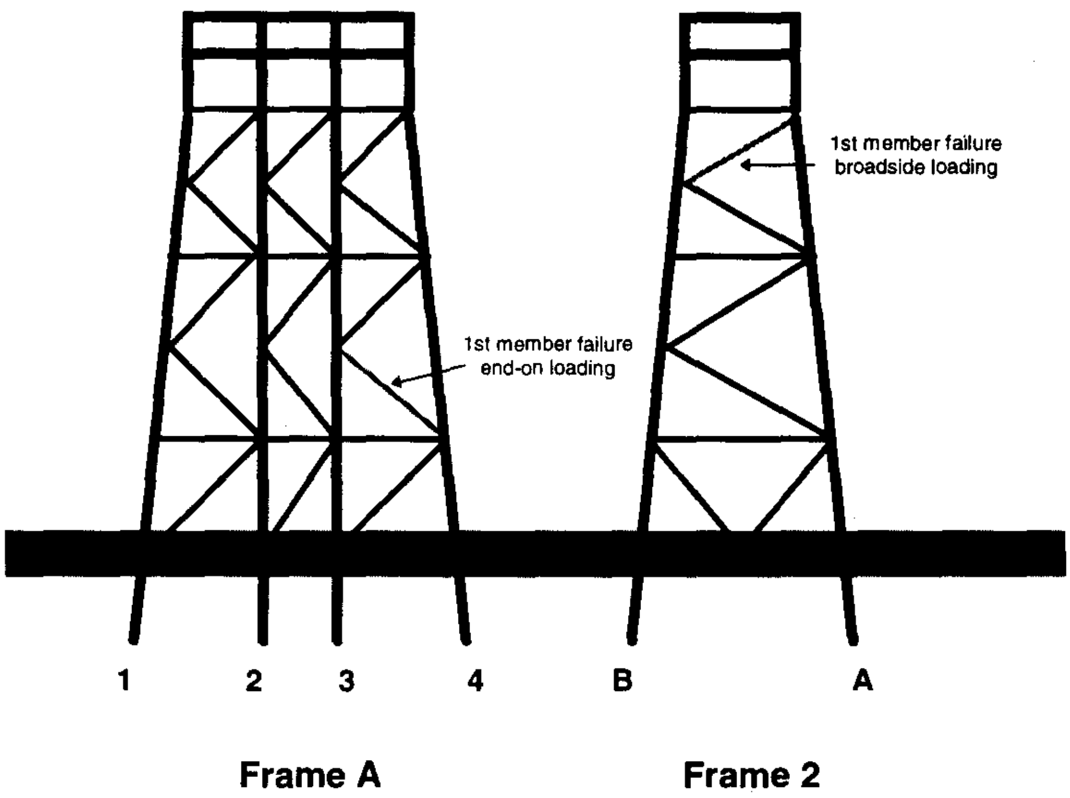

Figure 2.7.2-1: First Member Failure

3.0 COMPARING SUS AND ULSLEA

Results from the SUS analyses performed for this report indicate the approach to be somewhat conservative. For the PMB Benchmark platform, SUS indicates survival during Andrew is likely, and that platform did in fact survive. For SP62A and SS274A the results are less certain (further study would be needed), but it also appears both platforms would survive their historical extreme loading events. Failure is definitely indicated for ST151H, which reflects well with observed performance. However, SUS results for ST151K and ST161A indicate probable failure during Hurricane Andrew, which does not correlate with observed performance.

The results of the SUS evaluations of the six platforms studied are compared with results of similar studies using ULSLEA. For the purpose of numerical comparison, both the SUS and ULSLEA results are benchmarked against the results of analyses performed using USFOS. Finally, a discussion of the ease of application of the two approaches is made.

3.1 Comparisons with Results from USFOS

The results of both the SUS evaluations of the six subject platforms as well as ULSLEA analyses of the same platforms performed by Mortazavi and Bea (1996) are listed below in Table 3.1-1, along with analysis results from detailed evaluations performed by Loch and Bea (1995) and Stear and Bea (1996) using static pushover analysis (USFOS).

Both SUS and ULSLEA identify failure modes consistent with the ones identified by the USFOS analyses; however, SUS tends to provide a much more conservative (10-20% lower) estimate of ultimate strength than ULSLEA. Table 3.1-2 summarizes the ratios of the capacities predicted by SUS and ULSLEA to those predicted by USFOS (to establish the bias of the two simplified methods relative to a more accurate result). The mean bias for SUS/USFOS is 0.85; the coefficient of variation for SUS/USFOS is 11%. The mean bias for ULSLEA/USFOS is 0.95; the coefficient of variation for ULSLEA/USFOS is 7%. The simplified limit equilibrium analysis approach utilized within ULSLEA appears to be a closer and more consistent approximation to the results of more detailed analyses than the SUS approach.

Platform	Wave Direction	Simplified Ultimate Strength (SUS)		Limit Equilibrium Analysis (ULSLEA)		Static-Pushover Analysis (USFOS)	
		Failure Mode	Base Shear (kips)	Failure Mode	Base Shear (kips)	Failure Mode	Base Shear (kips)
PMB	End-On	5th jacket bay	3320	4th, 5th, 6th jacket bays	3200	5th, 6th jacket bays	3400
ST161A	End-On	1st jacket bay	2580	1st jacket bay	3100	1st jacket bay	3900
	Broadside	1st jacket bay	3120	1st jacket bay	3700	1st jacket bay	3900
ST151H	End-On	4th jacket bay	2300	4th jacket bay	2800	4th jacket bay	2700
	Broadside	2nd jacket bay	4300	2nd jacket bay	4200	2nd jacket bay	4500
ST151K	End-On	4th jacket bay	3920	3rd, 4th jacket bays	4500	4th jacket bay	4400
	Broadside	2nd jacket bay	4450	3rd jacket bay	4500	3rd jacket bay	4700
SP62A	End-On	4th jacket bay	6020	2nd, 3rd, 4th jacket bays	7000	3rd, 4th jacket bays	7440
	Broadside (W travel)	2nd jacket bay, jacket leg B3 in 6th bay	8640	2nd jacket bay	9200	2nd jacket bay	9510
SS274A	End-On	3rd jacket bay	3580	3rd jacket bay	4400	3rd, 4th jacket bays	4570
	Broadside	3rd jacket bay	4240	1st, 2nd, 3rd jacket bays	5000	1st, 2nd, 3rd jacket bays	5250

Table 3.1-1: SUS, ULSLEA, and USFOS Results for Six Platforms

Platform	Wave Direction	Ratio: SUS/USFOS	Ratio: ULSLEA/USFOS
PMB	End-On	0.977	0.941
ST161A	End-On	0.662	0.795
	Broadside	0.800	0.949
ST151H	End-On	0.852	1.037
	Broadside	0.956	0.933
ST151K	End-On	0.891	1.023
	Broadside	0.947	0.957
SP62A	End-On	0.809	0.941
	Broadside	0.909	0.967
SS274A	End-On	0.783	0.963
	Broadside	0.808	0.952
Mean		0.854	0.949
Standard Deviation		0.094	0.069
Coefficient of Variation		11%	7%

Table 3.1-2: Summary of Relative Bias in Capacity Predictions

3.2 Ease of Application

At a qualitative level, ULSLEA appears to provide superior analytical estimates of platform ultimate limit state. It should also be noted that to perform the ULSLEA evaluation required approximately 1-2 days' effort, whereas the modeling associated with the SUS evaluations took between 3-7 days. Much of this is due to the ease with which data may be assimilated into ULSLEA via the preprocessor.

Models used for SUS must be nearly as detailed as the models used when performing non-linear evaluations: the entire structure must be modeled. However, the main drawback to the SUS approach is the need to perform a series of analyses while incrementally increasing the wave height. This must be done, as the wave height causing failure is not known, and it is not good practice to use load factors in excess of unity with low wave heights when making capacity determinations, as it leads to concentration of load very low within the structure (failures of members in the upper portion of the jacket may be missed, as might a deck-leg failure mode). For each analysis, unity checks must then be made for all critical members, which is also time consuming. If the procedure could be automated, it would reduce the monitoring and evaluation time which must be devoted by engineers using the procedure, but it still represents a large time commitment for both personnel and computational facilities.

However, SUS is not without its advantages. Models generated for SUS may be used in more detailed evaluations should they be required, providing the input between the linear and non-linear analysis packages can be translated. Also, using the SUS approach allows for consideration of loads off the principle axes, should these cases be desired; however, these cases are more relevant to determining foundation failure (which rarely occurs in real life) as opposed to determining the strength of the structure.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Given that ULSLEA appears to be a more accurate tool with regards to estimating platform ULS, and the fact that ULSLEA is extremely user friendly, it appears to be the method of choice when considering the need for a quick and simple evaluation. However, it should be noted that neither of these simplified approaches has provisions for detecting local failures such as local buckling of jacket legs or fatigue; neither method would have identified the damage experienced by SS274A during Hurricane Hilda. Means of evaluating local failures of these types must be found.

Furthermore, much work remains in the area of estimating both foundation capacity and joint capacity. Use of API guidelines for requalification purposes results in extremely conservative estimates of joint and foundation performance, as identified by Vannan, et al. (1994), Loch and Bea (1995), and Stear and Bea (1996). Research effort should be devoted to developing criteria which can used as a better measure of service performance of these components.

Lastly, the issue of dynamic action has not been addressed. Loading rates can have substantial effects on the performance of these structures; the means to identify these effects and incorporate them within the simplified analysis framework certainly appears to be a worthwhile endeavor.

5.0 REFERENCES

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APPENDIX

ULSLEA EVALUATION OF SHELL SS274A

ULSLEA ANALYSIS OF SS274A

This appendix documents the ULSLEA evaluation of Shell SS274A. This platform has been described previously by Stear and Bea (1996); readers desiring more detailed information on the platform are referred to that report. This evaluation makes use of the latest version of ULSLEA, updated by Graduate Student Researcher James Stear 5/1/96.

INPUT DATA

Environmental Conditions

The following parameters were used for load generation within ULSLEA:

- water depth of 213 ft
- surge height of 3 ft
- wind velocity at 30 ft taken as 125 mph, with C_S=1.0
- wave height selected as 67 ft, with periods of 13.5 sec
- a constant current profile of 4 ft/sec
- C_D equal to 2.5 for the decks, and 1.2 for all tubular members
- 1.5 inches of marine growth for all submerged members
- current blockage of 0.8, with directional spreading of 0.88

Member Strength and Soil Properties

- yield stress of 43 ksi, with E = 29000 ksi
- k factor for braces of 0.65, with a residual strength factor of 1.0

Biases

- bias of 0.9 for loads

RESULTS

The analysis was intended to provide estimates of platform strength when subjected to the storm conditions listed above. Hence, no probabilistic risk assessment was performed. It should be noted that joint strength and pile capacities were not evaluated (dummy input was used); hence, the capacity profiles for both items should be ignored. Refer to the following figures for platform lateral load capacity.

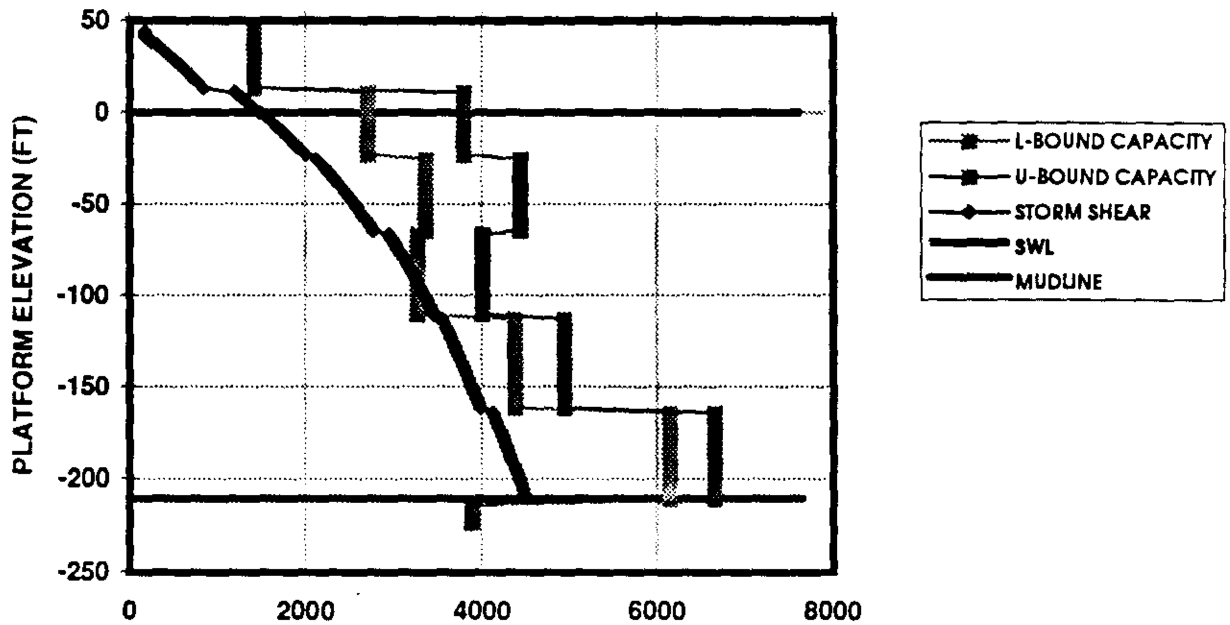

Figure A1: Platform Lateral Load Capacity (End-On Loads)

For the case of end-on loading, ULSLEA indicates SS274A can withstand a total lateral load of 4,400 kips, at which point members in the third jacket bay become at risk. For the case of broadside loading, ULSLEA indicates SS274A can withstand a total lateral load of 5,000 kips, with members in the first, second and third jacket bays at risk. These results are in good agreement with the results obtained using USFOS for the same platform (Stear, Bea, 1996).

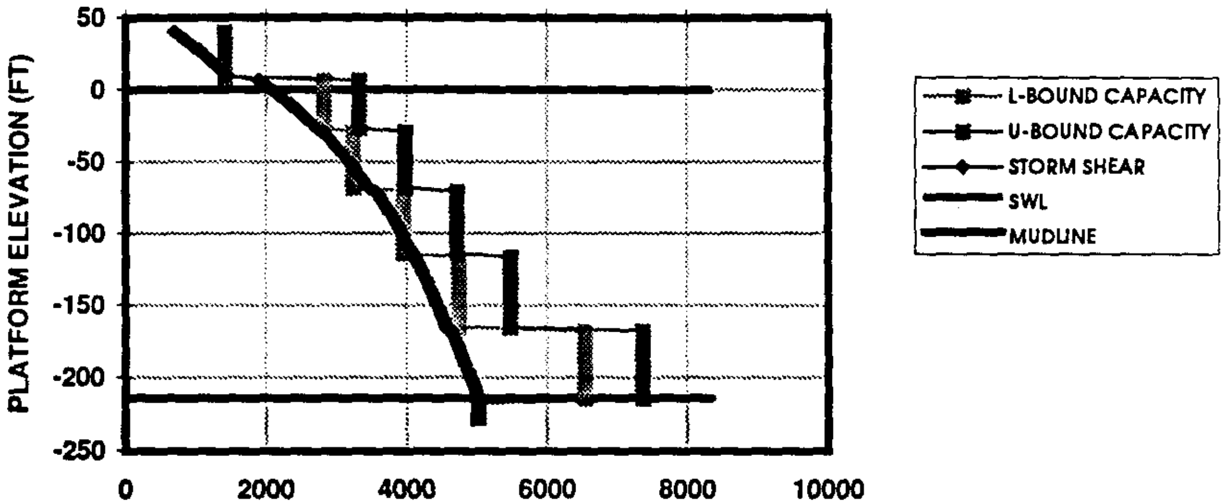

Figure A2: Platform Lateral Load Capacity (Broadside Loads)