Proceedings of the 5th (1995) International
OFFSHORE AND POLAR ENGINEERING CONFERENCE
The Hague, The Netherlands
June 11-16, 1995
ISOPE Paper No. 95-JSC-214

EVALUATION OF CAPACITIES OF TEMPLATE-TYPE GULF OF MEXICO PLATFORMS

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ABSTRACT

This paper details results from nonlinear analyses of the ultimate limit state performance characteristics of four Gulf of Mexico (GOM) platforms subjected to intense loadings from hurricane Andrew. These four platforms were located to the east of the track of hurricane Andrew, and were thus in the most intense portion of the storm [Smith, 1993]. The nonlinear analyses are able to replicate details of the observed behavior of the four structures. This replication is very dependent on realistic characterization of the performance characteristics of the pile foundations and on accurate information on the "as is" condition of the platforms before the storm.

INTRODUCTION

As part of a long-term research project, analysis procedures and computer programs are being developed that are intended to allow the engineer to make simplified, yet realistic evaluations of the dynamic, ultimate limit state behavior characteristics of conventional template-type offshore platforms subjected to storm loadings. A companion paper details the second-generation simplified procedures that have been developed to permit evaluations of storm loadings and static - cyclic capacities of such platforms [Bea, Mortazavi, 1995]. The first-generation approach and verifications have been described by Bea [1995] and DesRoches [1993]. The approach that is being developed to provide modifications that will permit the dynamic - transient loading effects to be taken into account has been described and initial results presented by Bea andYoung [1993].

The simplified procedures are being verified with results from complex nonlinear static and dynamic analyses that are able to provide details on the performance characteristics of platforms that are loaded to their ultimate limit state [Bea, DesRoches, 1993; Bea, Landeis, Craig, 1992; Bea, Craig, 1993].

This paper describes results from four platforms that have been analyzed as part of this research. These four platforms were located to the east of the track of hurricane Andrew, and were thus in the most intense portion of the storm [Smith, 1993]. The nonlinear analyses are able to replicate details of the observed behavior of the four structures. The remainder of this paper will detail the analyses and results for these four platforms.

PLATFORM 'B'

Platform 'B' (PB) is a self-contained, 8-leg, drilling and production platform with 12 well slots and 9 drilled wells (Figure 1). The platform was installed in 118 ft of water in the South Timbalier region in 1964. The platform was designed according to conventional 1963 criteria based on "25-year" return period design conditions (wave height of 55 feet).

Cellar and main deck elevations were located at + 34 ft. and + 47 ft. respectively. The major deck framing is 43 ft. by 93 ft. in plan, and the jacket legs are battered at one to eight in both broadside and end-on framing. The deck legs are 36 in. in diameter with a wall thickness of 0.625 in. and are connected to the tops of the piles with welded shim connections. The 39 in. diameter legs have an average wall thickness of 0.50 in. and have no joint cans. However, gusset plates are used with the jacket leg K-joints. The broadside braces vary from 14 in. in the first of four jacket bays to 20 in. in the lowest jacket bay, while the end-on bracing varies from 14 in. to 16 in.

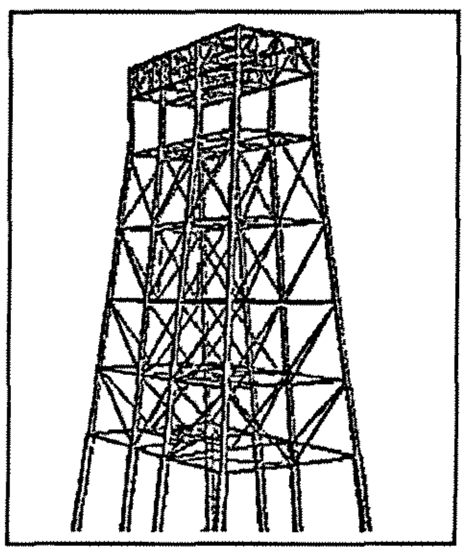

FIGURE 1: PLATFORM 'B'

Based on coupon tests performed after the platform was installed [Imm, et al., 1994], the jacket bracing and horizontal framing are made of nominal 50 ksi steel with an average yield strength of 58 ksi. The jacket legs and piles are composed of nominal 36 ksi steel with an average yield strength of

43 ksi. The strength of the legs and piles is based on the assumption that large members, i.e., greater than 30 in., were fabricated of plate steel, while the smaller members were constructed of rolled pipe sections.

The 36 in. piles extend 190 ft. below the mudline through 165 ft. of soft to stiff gray clay and 25 ft. of fine dense sand. At the time of design, anticipated pile loads were 770 tons in compression and 350 tons in tension. PB's piles were grouted inside its 39 in. jacket legs in 1973.

Although the platform has been subjected to several severe hurricanes (Carmen, 1974, and Andrew, 1992), PB has sustained no significant structural damage. This is due in part to previous platform remediations. In 1974, the eye of Hurricane Carmen passed within ten miles of PB. Cellar deck damage suggested the largest waves were approximately 58 ft from the southeast. Hindcast studies predicted slightly higher wave heights. Post-hurricane analyses indicated that the + 10 ft. vertical diagonal joints experienced compressive yielding. The platform was the subject of a risk analysis in 1988 that identified it as a significant risk [Imm, et al., 1994]. Consequently, in 1991 all eight conductors were removed and the cellar deck was cleared of all equipment.

In 1992, the eye of Hurricane Andrew passed within eight miles of the platform. Cellar deck damage suggested a maximum wave height between 60 ft. and 64 ft. from east-southeast, approximately fifteen degrees off broadside. Hindcast studies confirmed this observation. During this event, all four + 10 ft. K-joints in the broadside vertical trusses experienced yielding; two joints were at or close to their ultimate capacity. During the post-hurricane inspection, it was discovered that there was no grout in the pile-jacket leg annulus at + 10 ft. Below the water line, the grout performed well. If all four + 10 ft. K-joints yield a collapse mechanism is formed.

It is estimated that ten percent more lateral load would have collapsed the structure [Imm, et al., 1994]. Analyses showed that the load causing the joint yielding is very close to the load experienced during Andrew. More importantly, it was estimated that removing the conductors decreased the load during Andrew by twenty percent. Analyses also showed that the platform was capable of being re-loaded to the level experienced during Andrew. However, the + 10 ft. K-joints were grouted as an additional safety measure.

Several trials analyses were performed to find the wave height that caused platform failure with a load factor of unity. It was assumed that the majority of the load that could cause collapse of the platform was due to wave and current loads, and particularly wave-in-deck loads. The current and wind data from the Andrew hindcast studies were used and the wave height was varied. The wind forces used were based on hindcast conditions and current API RP 2A guidelines [API, 1993]. Boat landing, barge bumper, and loadings associated with other known appurtenances were simulated.

Hydrodynamic coefficients were chosen based on API guidelines [1993, 1994], recent test data [Bea, Pawsey, Litton, 1991; Heideman, Weaver, 1992], and engineering judgment. The best estimate drag and inertia coefficients (C_d, C_m) were as taken to be 1.2 for cylinders, respectively (all assumed to be hydrodynamically rough) [Rodenbush, 1986].

Based on the storm hindcast results [Cardone, Cox, 1992] and measured results from past GOM hurricanes [Bea, Pawsey, Litton, 1991], wave kinematics directional spreading factor equal to 0.88 was used for both the deck and jacket loads. A current blockage factor of 0.80 for broadside loading and 0.70 for end-on loading was also included. It should be noted that the wave height used for the end-on loading scenario did not create a load pattern that failed the platform with a load factor of unity. However, it was determined that this wave height was close to the realistic limit for this water depth.

The analytical model for PB contained the primary structural components of the platform. It was assumed that the main and cellar decks were not part of the first failure mode. Therefore, only the main framing members of the decks were modeled. The conductor framing was replaced with sufficiently rigid cross members to simulate their stiffness contribution. To account for a grouted pile-jacket leg annulus, the leg thickness was increased from 0.5 in. to 1.0 in. All members were given an initial imperfection, which was calculated by using Chen's buckling curve and member information for the critical braces in the structure [Chen, Ross, 1977]. This analysis was based on rigid joints.

The non-linear soil springs were developed using the PAR program assuming static loading [Bea, 1992]. Since analyses and post-Andrew inspections indicated that the first failure mode occurs in the upper jacket bay for both broadside and end-on loading, the exact performance of the soil springs is not critical in determining the ultimate lateral load resistance capacity of the platform. However, there are two items concerning the soil spring models that should be noted.

First, the T-Z (axial load - pile shaft displacement) and Q-Z (pile tip load - displacement) springs included as part of the model are linear as defined in the input to USFOS, which means that they will exhibit elastic behavior. Originally these nodes were defined using two force-displacement points, which translates into a straight line model. This strategy was intended to duplicate the approach used in the original analyses [Imm, et al, 1994]. When defining nonlinear soil properties, USFOS linearly extrapolates from the last two user defined points at both curve extremes. Therefore, since there were only two user defined points defining the non-linear behavior of the T-Z and Q-Z springs, USFOS extrapolated along the same original user defined line for both tension and compression behavior. The P-Y (lateral pile load - displacement) curves were defined using eight points, four points for each transverse direction. Thus the P-Y springs will exhibit nonlinear behavior.

As stated above, the linear elastic model of the T-Z and Q-Z springs will not significantly affect the determination the platform's ultimate capacity. However, this fact is based on the assumption that the pile-soil interaction is not part of the first failure mode. The ultimate pile uplift and compression forces were calculated. The largest tension and compression pile forces for both the broadside and end-on loading cases were lower than these previously calculated maximum values. Thus, the piles are not the weak link in the system for the load patterns used. Hence, while the ultimate capacity of the platform should not be effected by these linear spring, it is assumed that the shape of the displacement dependent results will not be exactly correct.

Secondly, the manner in which the combined T-Z and P-Y springs were modeled is prone to potential error, especially for large displacements. Again, this error is assumed not to affect the ultimate capacity of the platform, but it does cause inaccuracies that are worth mentioning. In the PB model the T-Z and P-Y were combined into a two node nonlinear soil spring. The combined spring has T-Z spring properties for its axial displacements and P-Y spring properties for its transverse displacements. Both axial and transverse displacements are measured relative to the original coordinates of the element's end nodes. However, when a T-Z / P-Y soil spring element becomes deformed the relative position of the two end nodes must be considered for the deformed shape. Since this is not the case, in a deformed position the displacement transverse to the element will be resisted by the P-Y spring and the T-Z spring. The exact spring properties for any given deformed shape can be solved using vector analysis.

Broadside Loading

The force-displacement curve for broadside loading is shown in Figure 2. This curve indicates that platform fails at 0.907 of the reference load pattern or a total base shear of 3,860 kips. This lateral loading capacity is less than the 4,900 kips reported by Imm et al. [1993]. This difference is due to the differences in the loading patterns utilized in the two analyses. The analyses reported in this paper have larger wave forces acting on the platform lower deck.

Figure 2 indicates that the platform has no reserve strength after the first brace - joint failure. However, it is important to note that the platform can experience large inelastic displacement before a failure mechanism is formed. If the force-displacement curve were extended it would show that eventually, the jacket legs develop sufficient resistance in bending to cause buckling of the braces in the third jacket bay.

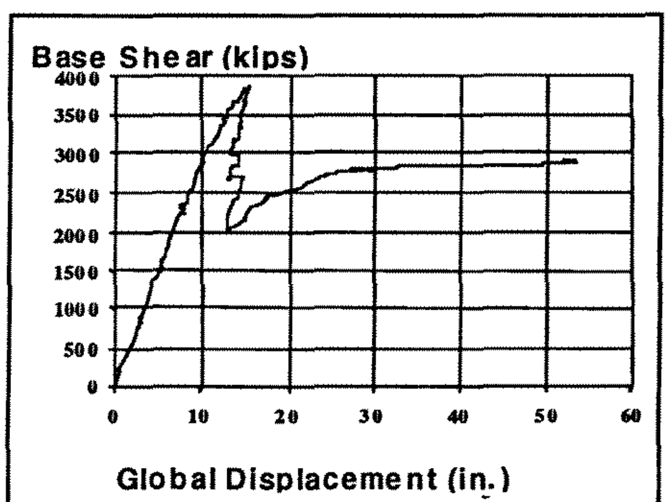

FIGURE 2: BROADSIDE FORCE-DISPLACEMENT RELATIONSHIP

End-on Loading

The force-displacement curve for end-on loading is shown in Figure 3. This curve indicates that the uppermost compression braces buckle at 1.12 of the reference load pattern or a total base shear of 3,900 kips. Figure 4 shows that after the compression braces - joints in the fourth jacket bay fail the platform has a small increase in resistance capacity until the compression braces in the third jacket bay and the horizontal framing between these two levels almost simultaneously fail, at which point the platform is at imminent collapse.

Comparisons of Analytical and Observed Results

The hurricane hindcast data [Cardone, Cox (1992] and observed platform performance indicate that PB survived 60 - 64 ft waves 15 degrees off of broadside during hurricane Andrew. Approximately 96 percent the peak loading developed during the storm was resisted by the broadside framing. The USFOS analysis indicates that the platform experiences first significant member failure, brace - joint failure, at 91 percent of the load from a 64 ft direct broadside wave.

The wave deck loads are very significant for this loading profile. The deck loads represent nearly 40 percent of the total load. This is in agreement with the results documented by Imm, et al. [1994]. The hydrodynamic loads are highly sensitive to the wave height and the surge height. In addition, initial imperfection magnitude and direction are realistic but somewhat conservative. Hence, the brace - joint failure load represents a probable lower bound estimate of the true brace strength. This same result was observed by Imm et al. [1993] based on results from K-braced frame tests.

Taking the above factors into consideration, the USFOS results indicate that PB should survive the loads from hurricane Andrew. The analytical results are in conformance with the observed performance of PB after hurricane Andrew [Imm, et al., 1994].

PLATFORM 'C'

Platform 'C' (PC) (Figure 4) was installed in the GOM Ship Shoal region in 1970. This platform is a self contained four pile drilling and production platform located in 157 ft. of water. PC survived hurricane Andrew without significant damage.

The platform has four conductors and eight risers. The PC decks are located at elevations of +33 ft., +43 ft., +56 ft. and +71 ft. The deck legs form a 30 ft. by 30 ft. square in plan and the jacket legs are battered at 1:11 in both primary directions.

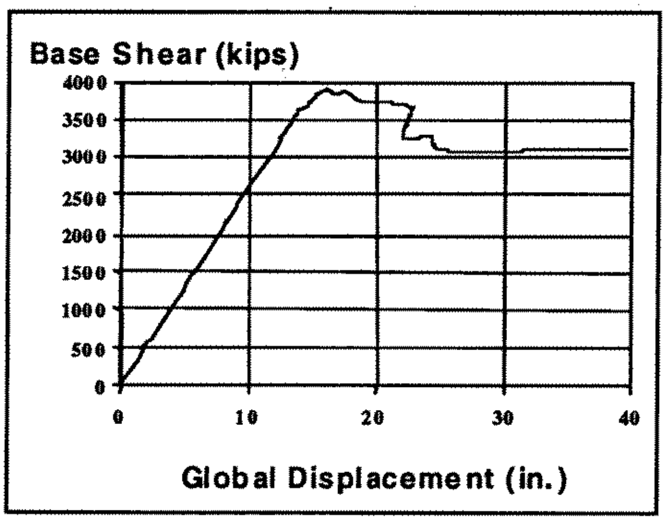

FIGURE 3: END-ON FORCE-DISPLACEMENT RELATIONSHIP

The piles for PC run through the jacket legs, but unlike PB the pile-jacket leg annulus is not grouted. The 36 in. diameter piles extend 355 ft. below the mudline through 328 ft. of soft to stiff gray clay and 27 ft. of fine dense sand. The sand layer starts at 197 ft. below mudline. The clay above the sand is generally soft and silty, while the clay below the sand is stiff.

While the pile-leg annulus is not grouted, the jacket legs and most other intersecting members have joint cans. The 39.5 in. diameter jacket legs are 0.5 in. thick while the joint cans are 1.25 in. thick. The deck legs are 36 in. in diameter with a wall thickness of 1.25 in. and are connected to the tops of the piles. The vertical braces vary from 16 in. in the top or seventh jacket bay to 20 in. in the first jacket bay. All members reportedly are constructed of nominal 36 ksi steel with an average yield stress of 43 ksi.

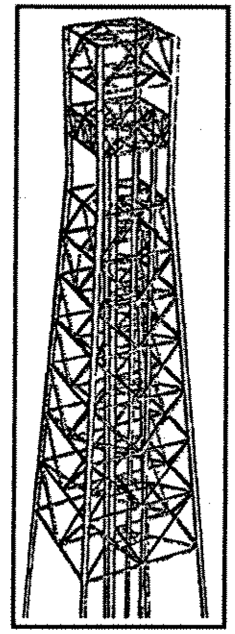

FIGURE 4: PLATFORM 'C'

Many know PC as the "PMB Benchmark Platform" [PMB Engineering, 1994]. PC was used as a test structure for a Joint Industry Project (JIP). The JIP's main objective was to assess the variability in the calculated ultimate capacity of a typical fixed offshore platform due to different assumptions, different code interpretations, different software packages, and human error. The JIP participants were to strictly use API guidelines [1993, 1994] to define the loading and capacity parameters of the analyses. However, the software and analysis techniques used varied between companies. Analysis results specified by PMB were submitted by all the participants. These results were then compared to assess their variability [PMB Engineering, 1994].

The platform was analyzed with foundation simulations based on 'static' and 'dynamic' pile - soil interaction characteristics [Bea, 1987; 1992a]. The static pile simulations were based on the soil boring test results (wireline samples, undrained - unconsolidated triaxial tests) and API static pile capacity guidelines [API, 1993]. The dynamic pile simulations were based on soil broing test results corrected for sample disturbance [Quiros, et al., 1983] and dynamic pile capacitiy guidelines in the API *Commentary on Pile Capacity for Axial Cyclic Loadings* [1993]. The differences between static and dynamic axial and lateral pile capacities ranged from 2 to 3 [Bea, 1987]. The differences between static and dynamic axial and lateral pile stiffnesses were as great as 10. These results are in agreement with those developed by Tang [1988, 1990] These results also are justified by comparisons of static and dynamic field pile load test data [Bea, Audibert, 1979; Bea, 1980; Bea, et al., 1984].

Wind forces were calculated using API RP 2A guidelines [1993]. Appurtenance and deck loads were calculated by hand using the wave kinematics developed in WAJAC. The broadside and end-on loading scenarios are essentially identical and thus, only one direction was analyzed.

As with PB, hydrodynamic coefficients were chosen based on recent test data and engineering judgment. The, the best estimate drag and inertia coefficients were taken to be 1.2 for cylinders. A wave kinematics factor equal to 0.88 was used for both the deck and jacket loads. A current blockage factor of 0.80 was also included.

The computer model contains the primary structural components of the platform. It was assumed that the main and cellar decks were not part of the first failure mode. Therefore, only the main framing members of the decks were modeled. The conductors were transversely slaved to nearby nodes in the horizontal framing from the first deck down to the mudline. The piles were transversely slaved to the jacket legs that they run through except at the top, where the piles, jacket legs and deck legs are rigidly connected at all four corners. All members were given an initial imperfection, which was chosen based on the API standards for allowable pre-construction member imperfections. Finally, since the platform contains joint cans this analysis used rigid joints.

Single node non-linear soil springs were developed using the procedures outlined API RP 2A [1993]. These procedures assumed static and dynamic loading assumptions [API, 1993; 1994]. Based on results from past analyses of GOM platforms subjected to hurricane loadings, pile simulations based on traditional static pile capacity methods can be too conservative is some cases and will indicate a false failure in the foundation [Bea, DesRoches, 1993].

Loading Results

The initiating failure mode for PC based on the static pile characterization is pile plunging. The force-displacement history for broadside loading is shown in Figure 5. This curve indicates that platform fails at 0.628 of the reference load pattern or a total base shear of 1,700 kips at a displacement of about 24 in. From Figure 5 it can be seen that the platform has a constant stiffness after all the T-Z and Q-Z springs of the compression piles have reached their final plateaus

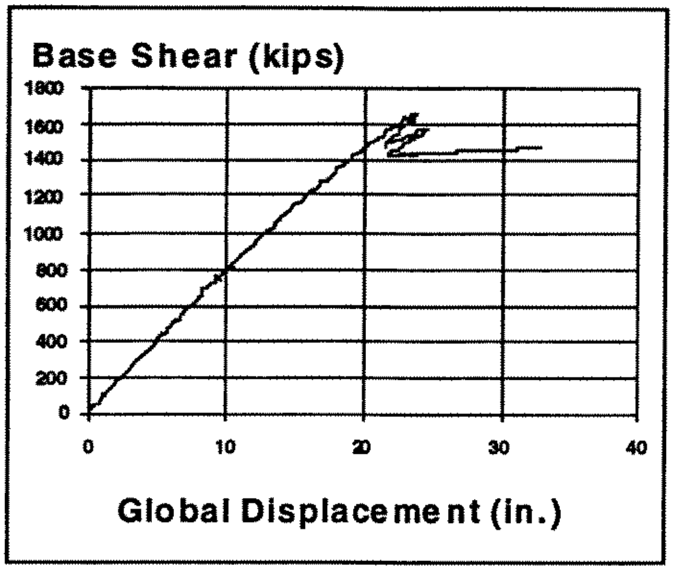

FIGURE 5 - FORCE - DISPLACEMENT RELATIONSHIP BASED ON STATIC PILE CHARACTERISTICS

Since the foundation was shown to be the weak link in the platform, an analysis based on a dynamic pile characterization was also performed.. Results from the USFOS analyses showed that if the foundation was characterized based on consideration of dynamic effects, the braces in the second jacket bay became the weak link. The second bay compression braces buckled at 1.30 of the reference load pattern or a total base shear of 3,440 kips. After the compression braces in the second jacket bay buckled, the braces in the third jacket bay buckled and the jacket began to "unzip".

The lateral force - displacement characteristics for the analyses based on the dynamic pile characterization is given in Figure 6. The peak lateral load capacity is now 3,500 kips and it is reached at a lateral displacement of about 9 in. Based on the dynamic pile characterization, the lateral load capacity of the platform is about doubled.

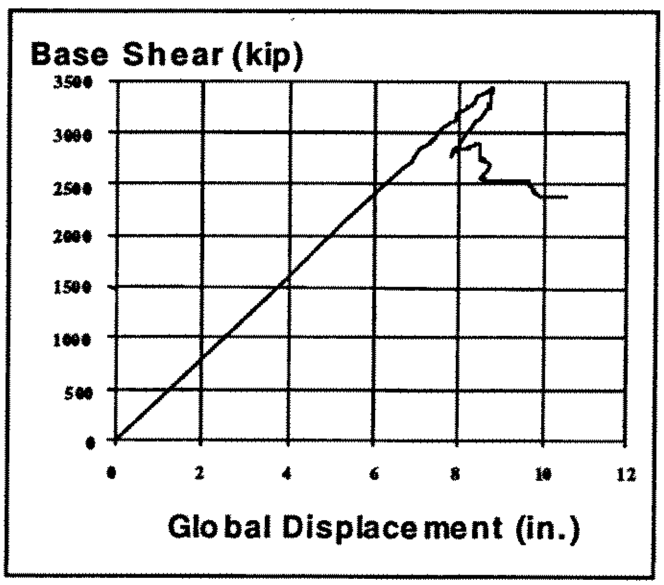

FIGURE 6: FORCE - DISPLACEMENT RELATIONSHIP FOR DYNAMIC PILE CHARACTERIZATION

Comparison of Analytical and Observed Results

The hurricane hindcast data [Cardone, Cox (1992] and observed platform performance indicate that PC survived 53 - 56 ft waves during hurricane Andrew. Based on the results from the analyses performed on PC, the total lateral loading associated with these conditions ranged from 1,700 to 1,900 kips. These loadings exceed the platform capacity that was based on static pile capacities. However, they do not exceed the platform capacity that was based on dynamic pile capacities. Given that the platform survived hurricane Andrew without significant damage, it is concluded that the platform capacity based on the dynamic pile simulations is more realistic.

For broadside or end-on loading, the range in the PMB benchmark lateral load capacities was 1,500 kips to 3,600 kips [PMB, 1994]. Based on the analyses performed during this study, the lower bound results were obtained when the static pile capacity was utilized (Figure 5) and the upper bound when the dynamic pile capacity was utilized (Figure 6). There is good agreement between these two sets of results. The majority of the range between the lower bound and upper bound results is attributable to differences in how the foundation is simulated.

WELLHEAD PROTECTORS 1 AND 2

The eye of hurricane Andrew passed within a few miles to the west of Wellhead Protectors 1 (WP1) and 2 (WP2) (Figures 7 and 8). Hurricane Andrew produced extreme storm loadings which caused WP1 to collapse. Diver surveys made after the storm indicated WP1 failed by pull out of the piles on the south side of the platform. The seemingly identical WP2 did not collapse; there was no significant damage to this structure. The goal of this study was to determine how the forces developed by hurricane Andrew could have caused the collapse of WP1 and not the collapse of WP2.

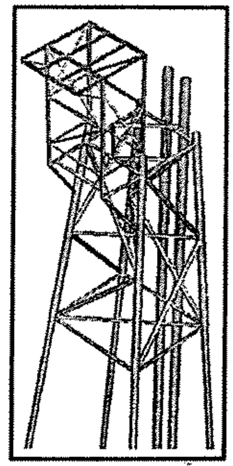

FIGURE 7: WELLHEAD PROTECTOR 1

The study of WP 1 and WP2 involved the use of three computer programs: 1) StruCad*3D, 2) USFOS, and 3) ULSLEA. StruCad*3D [Zentech, 1993] was to perform linear elastic analyses in order to gain an overall understanding of the response of the two structures to storm loading. ULSLEA (Ultimate Limit State Limit Equilibrium Analyses) [Mortazavi, Bea, 1994] is a technique which performs simplified analyses of the load resisting capacities of offshore template structures [Bea, Mortazavi, 1995]. This approach serves as a link between linear and nonlinear analyses by providing estimates of the storm loads required to cause first yield and collapse of the wellhead protectors. The third approach utilized the nonlinear analysis program USFOS [SINTEF, 1994] to perform static pushover analyses of the wellhead protectors. *In this paper, because of space limitations we will discuss only the results from the USFOS analyses.* A future paper will detail the results from the other two methods and compare these results.

Structural Characteristics

The two structures evaluated herein were both located in the South Timbalier area. The two wellhead protectors were designed and installed early in the 1980's by the same firm. The two wellhead protectors were designed according to the same API RP 2A guideline. The slightly older WP1 is located in 52 ft. of water and is oriented 45° counterclockwise from true north. WP2 is located in slightly shallower water (49 ft.) and is oriented parallel to true north. Both structures are two bay, four pile template structures designed to provide limited facilities for 36 in. diameter caisson well risers (Figures 7 and 8). Both protectors have offset braced helipads and boat landings for easy access.

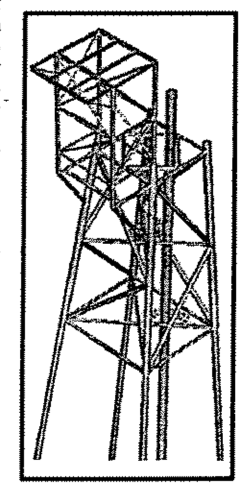

FIGURE 8: WELLHEAD PROTECTOR 2

The jacket framing of the two structures is almost identical, with WP2 having slightly smaller diameter jacket legs and piles; 28 and 24 in.; as opposed to 30 and 26 in. Diagonal vertical bracing is made up of 18 in. tubulars, while plan bracing is composed of 12.75 in. tubulars on all three levels. All members were fabricated using A36 grade steel.

The most prominent difference between the two structures, other than water depth and orientation, lies in the number and location of caisson risers each structure must support. The two caissons of WP1 are located just outside of the structure north end of the jacket and are not tied substantially to the jacket. WP2's caisson is rigidly framed within the interior of the jacket.

Soil and Foundation Characteristics

The foundations for the two structures are very similar only in that they are both composed of four piles. The design of these piles is quite different. WP1's piles are 187 ft. long, 26 in. in diameter and are comprised of several segments. At the tip there is a five foot pile shoe with 0.75 in. thick walls for driving. It is followed by 100 ft. of 0.5 in. thick walls. Above this segment is the only pile splice found below the mudline. Here the wall thickness increases again to 0.75 in. for another ten ft. The remainder of the pile above the pilehead and into the lower bay is 1.125 in. thick. WP2's piles are slightly longer (190 ft.) than those of WP1 to compensate for its smaller diameter of 24 in. It's upper wall thickness are generally larger as well, running at 1.213 in. to withstand the large bending stresses found in the piles near the mudline. The remaining distribution is essentially the same as for WP1.

Nonlinear axial soil curves were generated from soil boring tests [Law 1981]. The soil conditions were reported as consisting of a deep 172 ft. layer of soft clays overlying a deep layer of stiff sand. Shear strengths of the clay run between 0.31 ksf at the surface to 0.5 ksf at a depth of 64 ft., and to 1.5 ksf at the sand layer boundary. It was recommended that the structures' piles should be designed so as to be driven to depth into the sand in order to take advantage of its high compressive bearing capacity.

The pile - soil interactions were modeled using API RP 2A guidelines for static (T-Z, Q-Z) - cyclic (P-Y) and dynamic loading conditions [API, 1993; 1994].

The results of the study based on StruCad*3D and ULSLEA initially indicated that WP1 and WP2 should have behaved similarly; both should have survived. At this point, the pile driving records for the structures were obtained and reviewed. It was discovered that both of the piles on the south side of WP1 had been under-driven by 5 feet. All of the piles in WP2 had been driven to their design penetrations.

Storm Loadings

Wind, wave and current characteristics were chosen from environmental data provided from the Hurricane Andrew hindcast [Cardone and Cox 1992]. The structure was loaded along its principal axes. The following hydrodynamic parameter were used in these analyses:

Wind: 98 knots; ABS wind profile
Wave: 40 ft. height; 9.5 second period
9th Order Stream Function Wave Theory
Current: 6 fps constant over depth
Surge and Tide: 3 ft.
Drag Coefficient (tubular members): Cd = 1.2

The two structures were loaded only along their principal axes to provide consistency between the various approaches employed to analyze structural response. Wave loads for USFOS were generated by the seastate program WAJAC [DNV 1993] which determines peak loads using phase angle intervals of 1°. The global base shears developed on WP1 and WP2 during the passage of Andrew are summarized in Figure 9. The results indicate WP1 experienced peak lateral loadings that were about 20 % larger than those on WP2. The peak lateral loading on WP1 was 1,100 kips and on WP2 was 850 kips.

Push-Over Results

The static push-over results for WP1 and WP2 based on the USFOS results are summarized in Figure 10. The "double humps" found in both analyses result from the increased stiffness of the structures when contact between the jacket and caissons occur. The negative stiffness found at the end of all analyses represents pile pullout. The large lateral deformations produce plastic hinges in the piles which produce a near mechanism. It is the additional strength and rigidity of the caissons which prevents the structures from soft story collapse. This added stiffness allows the full axial capacity of the soils to be exceeded to produce pile pullout. The maximum lateral load capacity of WP1 is 910 kips and WP2 880 kips.

The USFOS result that both structures fail due to pile yielding and pullout was confirmed by results from the StruCad*3D and ULSLEA analyses. The ratio of the peak lateral loading during hurricane Andrew to the maximum lateral loading capacity is 1.2 and 0.95 for WP1 and WP2, respectively. The analyses indicate that WP1 should have failed due to pile pullout and WP2 should have survived. The paradox of why these two seemingly identical structures behaved differently was due to the differences in the appurtenances (well conductors), the manner in which the wells were tied into the structures, and the under-driven piles. The effects of these differences only became evident when these "details" were determined and their implications integrated into the analyses.

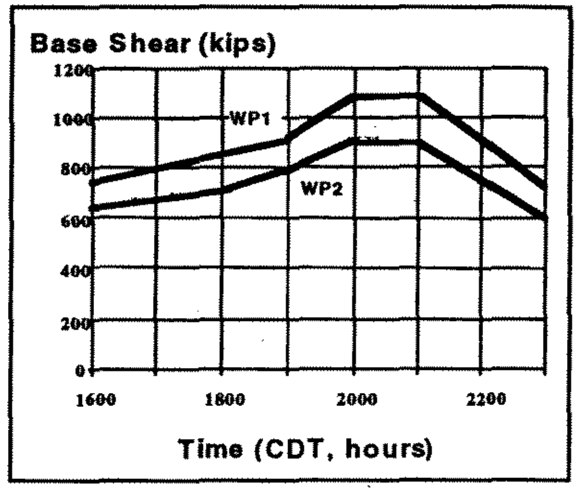

FIGURE 9: TIME HISTORY OF MAXIMUM BASE SHEARS ON WP1 AND WP2 DURING HURRICANE ANDREW

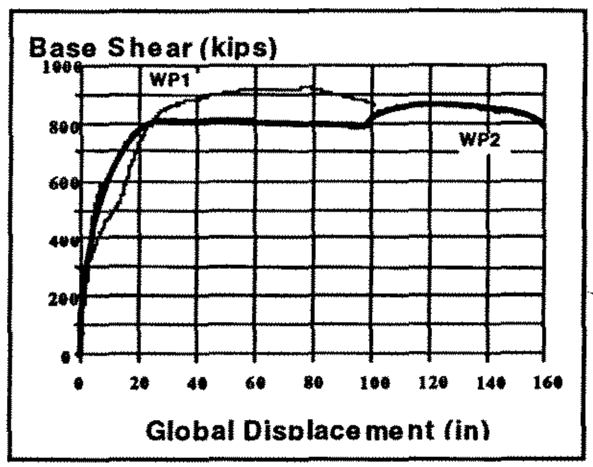

FIGURE 10: LOAD - DISPLACEMENT CHARACTERISTICS FOR WP1 AND WP2

CONCLUSIONS

This paper details results from nonlinear analyses of the ultimate limit state performance characteristics of four Gulf of Mexico Platforms subjected to intense loadings from hurricane Andrew. One of the platforms (platform 'B') is a conventional 8-leg drilling and production platform that survived the loadings developed during hurricane Andrew. Inspections of this platform following the storm disclosed severe damage to the joints and braces that indicated that the platform was loaded nearly to its ultimate limit state lateral load capacity. The analyses are able to replicate this performance

Two of the other platforms are 4-leg well protectors that also survived hurricane Andrew. One of these platforms (platform 'C') was the subject of an industry study in which a large number of engineering organizations were provided identical information on the platform and requested to determine the loadings and capacities of the structure [PMB Engineering Inc., 1994]. This platform survived hurricane Andrew without significant damage. The analyses indicate that it should have performed in this manner. The analyses indicate that the very large range in structure capacities obtained is due principally to differences in the procedures used to simulate the pile foundation performance characteristics. Traditional 'static' characterizations form a lower bound while 'dynamic' characterizations form an upper bound for the lateral loading capacities of this particular structure.

The third 4-leg well protector (WP1) was located directly in the path of hurricane Andrew. It collapsed. The fourth nearby, seemingly identical 4-leg well protector (WP2) was not damaged. The analyses are able to explain this paradox. When subtle differences in the appurtenances, well attachments, and foundation piling penetrations were recognized, the analytical results indicated that the platform that survived should have and the platform that collapsed should have. It was not 'probabilistic' differences that resulted in the survival and collapse, but rather 'deterministic' differences. This experience indicates that observed failures and survivals of platforms can provide useful information when the details of the structures are known. When platforms are loaded at or close to their collapse capacity, nuances in their elements, loadings, and performance can determine the difference between survival and failure. Analyses performed on structures without these details can provide misleading results [Puskar, et al., 1994].

ACKNOWLEDGMENTS

This research has made extensive use of the software USFOS provided by SINTEF (Trondheim, Norway) and the SESAM pre and post processors for USFOS provided by Det Norske Veritas. (DNV, Oslo, Norway and Houston, Texas). The implementation and initial application of this software on a RISC 6000 365 was performed by a visiting research scholar from SINTEF, Mr. Øyvind Hellan. The assistance, guidance, and instruction provided by SINTEF, DNV, and Mr. Øyvind Hellan are greatfully acknowledged.

The results summarized in this paper have been developed from joint industry - government sponsored projects conducted during the past three years. Appreciation is expressed to the sponsors including Arco Exploration Co., Exxon Production Research Co., UNOCAL Corp., Shell Oil Co., Mobil Research and Development Corp., the U. S. Minerals Management Service, the California State Lands Commission, and the California and National Sea Grant College Programs. Support and assistance have also been provided by Chevron Petroleum Technology Co. and Amoco Production Co.

This paper is funded in part by a grant from the National Sea Grant College Program, National Ocanic and Atmospheric Administration, U. S. Department of Commerce, under grant number NA89AA-D-SG 138, project numbers R/OE-11 and R/OE-19 through the California Sea Grant College, and in part by the California State Resources Agency. The views expressed herein are those of the authors and do not necessarily reflect the views of NOAA or any of its sub-agencies. The U. S. Government is authorized to reproduce and distribute for governmental purposes.

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