R. G. Bea

K. J. Loch

P. L. Young

Department of Civil and Environmental Engineering, and
Department of Naval Architecture and Offshore Engineering,
University of California,
Berkeley, CA 94720-1712

Capacities of Template-Type Platforms in the Gulf of Mexico During Hurricane Andrew

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 limit equilibrium-based procedures that have been developed to permit evaluations of storm loadings and static-cyclic capacities of such platforms (Bea and Mortazavi, 1996).

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 platforms include one eight-leg and one four-leg drilling and production structure, and two four-leg well protectors. These structures are identified as platforms "B" (PB), "C" (PC), WP 1, and WP 2.

The structures were analyzed using nonlinear, static, pushover analyses. The nonlinear finite element computer program USFOS was utilized to perform the analyses (SINTEF, 1994; Hellan et al., 1993, 1994). Up to the first member failure in the structures, the USFOS analyses were load-controlled. Thereafter, they were displacement-controlled. Wave and wind loads in the deck were calculated and applied as nodal loads. The hydrodynamic forces on jacket were generated using the WAJAC wave load program (DNV, 1993). Stokes fifth-order wave theory was used and member loads were calculated based on the API RP 2A guidelines (1993).

None of the results reported herein have incorporated corrections to recognize wave dynamics-platform nonlinear response characteristics (Bea and Young, 1993). For the structures discussed in this paper, these corrections are indicated to increase the static lateral loading capacities by factors in the range of Fv = 1.0 to 1.2 (Fv = dynamic lateral loading capacity/static loading capacity).

Platform B

PB is a self-contained, eight-leg, drilling and production platform with twelve well slots and nine drilled wells (Fig. 1). The platform was installed in 118 ft (36 m) of water in the South Timbalier region in 1964. The platform was designed according to conventional 1963 criteria based on "25-yr" return period design conditions (wave height of 55 ft, 17 m).

Cellar and main deck elevations were located at +34 ft (10 m) and +47 ft (14 m), respectively. The major deck framing is 43 ft (13 m) by 93 ft (28 m) 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. (91 cm) in diameter with a wall thickness of 0.625 in. (1.6 cm) and are connected to the tops of the piles with welded shim connections. The 39-in- (99-cm-) dia legs have an average wall thickness of 0.50 in. (1.3 cm) and have no joint cans. However, gusset plates are used with the jacket leg K-joints. The broadside braces vary from 14 in. (36 cm) in the first of four jacket bays to 20 in. (51 cm) in the lowest jacket bay, while the end-on bracing varies from 14 in. (36 cm) to 16 in. (41 cm).

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 (345 MPa) steel with an average yield strength of 58 ksi (400 MPa). The jacket legs and piles are composed of nominal 36 ksi (248 MPa) steel with an average yield strength of 43 ksi (297 MPa). The strength of the legs and piles is based on the assumption that large members, i.e., greater than 30 in. (76 cm), were fabricated of plate steel, while the smaller members were constructed of rolled pipe sections.

The 36-in. (91-cm) piles extend 190 ft (58 m) below the mudline through 165 ft (50 m) of soft to stiff gray clay and 25 ft (7.6 m) of fine dense sand. PB piles were grouted inside its 39-in. (99-cm) jacket legs in 1973.

Although the platform has been subjected to several severe hurricanes (Carmen in 1974 and Andrew in 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 10 mi (17 km) of PB. Cellar deck damage suggested the largest waves were approximately 58 ft (18 m) from the southeast. Hindcast studies predicted slightly higher wave heights. Post-hurricane analyses indicated that the +10-ft (3-m) 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 8 mi (13 km) of the platform. Cellar deck damage suggested a maximum wave height between 60 ft (18 m) and 64 ft (20 m) from east-southeast, approximately 15 deg off broadside. Hindcast studies confirmed this observation. During this event, all four +10-ft (3-m) K-joints in the broadside vertical trusses experi-

Contributed by the OMAE Division for publication in the JOURNAL OF OFFSHORE MECHANICS AND ARCTIC ENGINEERING. Manuscript received by the OMAE Division, October 16, 1995; revised manuscript received February 27, 1996. Technical Editor: S. K. Chakrabarti.

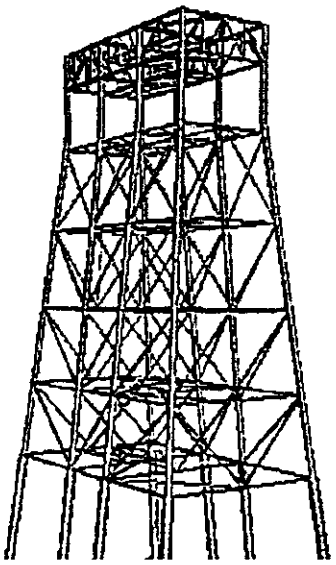

Fig. 1 Platform B

enced 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 (3 m). Below the waterline, the grout performed well. If all four +10 ft (3 m) K-joints yield, a collapse mechanism is formed.

It was 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 was very close to the load experienced during Andrew. More importantly, it was estimated that removing the conductors decreased the load during Andrew by 20 percent. Analyses also showed that the platform was capable of being reloaded to the level experienced during Andrew. However, the +10-ft (3-m) K-joints were grouted as an additional safety measure.

Several trial 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 et al., 1991; Heideman and Weaver, 1992), and engineering judgment. The *best-estimate* drag and inertia coefficients 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 and Cox, 1992) and measured results from past GOM hurricanes (Bea et al., 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 (1.3 cm) to 1.0 in (2.54 cm). 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 and Ross, 1977). The direction of the member imperfection was chosen so that the moments induced by axial loads would be additive to the local wave-force-induced moments. This analysis was based on rigid joints with appropriate consideration of the joint tensile and compressive strengths.

The nonlinear soil springs were developed using the PAR (pile-soil analysis routines) 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 translate 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 nonlinear 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 in the foregoing, 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.

Broadside Loading. The force-displacement curve for broadside loading is shown in Fig. 2. This curve indicates that platform fails at 0.907 of the reference load pattern or a total

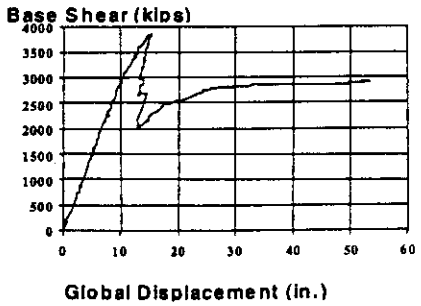

Fig. 2 Platform B broadside loading force-displacement relationship (1 kip = 4.4 kN, 1 in. = 2.5 cm)

base shear of 3860 kips (17 MN). This lateral loading capacity is less than the 4900 kips (22 MN) 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.

End-on Loading. The force-displacement curve for end-on loading is shown in Fig. 3. This curve indicates that the uppermost compression braces buckle at 1.12 of the reference load pattern or a total base shear of 3900 kips (17 MN). Figure 3 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 and Cox, 1992) and observed platform performance indicate that PB survived 60-ft (18-m) to 64-ft (20-m) waves 15 deg off the 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 (20-m) 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. The hydrodynamic loads are very sensitive to the wave height and crest elevation, 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 foregoing 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

PC (Fig. 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 (48 m) of water. PC survived hurricane Andrew without significant damage.

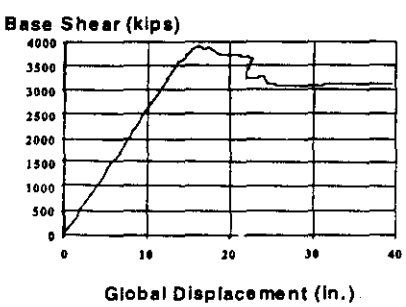

Fig. 3 Platform B end-on loading force-displacement relationship (1 kip = 4.4 kN, 1 in. = 2.5 cm)

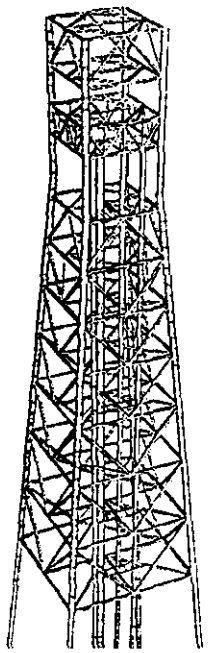

Fig. 4 Platform C

The platform has four conductors and eight risers. The PC decks are located at elevations of +33 ft (10 m), +43 ft (13 m), +56 ft (17 m) and +71 ft (22 m) The deck legs form a 30-ft (9-m) by 30-ft (9-m) square plan at the top of the jacket, and the jacket legs are battered at 1:11 in both primary directions.

The piles for PC run through the jacket legs, but unlike PB, the pile-jacket leg annulus is not grouted. The 36-in- (91-cm-) dia piles extend 355 ft (108 m) below the mudline through 328 ft (100 m) of soft to stiff gray clay and 27 ft (8 m) of fine dense sand. The sand layer starts at 197 ft (60 m) 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-(100-cm-) dia jacket legs are 0.5 in (1.3 cm) thick, while the joint cans are 1.25 in. (3.2 cm) thick. The deck legs are 36 in. (91 cm) in diameter with a wall thickness of 1.25 in. (3.2 cm) and are connected to the tops of the piles. The vertical braces vary from 16 in. (41 cm) in the top jacket bay to 20 in. (51 cm) in the bottom jacket bay. All members reportedly are constructed of nominal 36 ksi (248 MPa) steel with an average yield stress of 43 ksi (297 MPa).

PC was used as a test structure for a joint industry project (JIP) (PMB Engineering, 1994; Digre et al., 1995). The JIP's main objective was to assess the variability in the calculated maximum lateral loadings and ultimate capacity of a typical fixed offshore platform. 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; Digre et al., 1995).

In this study, 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 (1993) static pile capacity guidelines. The dynamic pile simulations were based on soil boring test results corrected for sample disturbance (Quiros et al., 1983) and dynamic pile capacity guide-

lines 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 stiffness 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 and Audibert, 1979; Bea, 1980; Bea et al., 1984) and analyses of the performance characteristics of platforms in Gulf of Mexico hurricanes (Bea and DesRoches, 1993; Bea and Craig, 1993).

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 (Det Norske Veritas, 1993). The broadside and end-on loading scenarios are essentially identical, and, consequently, only one direction was analyzed.

As with PB, hydrodynamic coefficients were chosen based on recent test data and engineering judgment. The best-estimate hydrodynamic drag and inertia coefficients for rough cylinders were taken to be 1.2 for cylinders (Rodenbusch, 1986). A wave kinematics factor equal to 0.88 was used for both the deck and jacket loads. A current blockage factor of 0.80 also was 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 preconstruction member imperfections. As in the PB analyses, the direction of the member imperfection was chosen so that the moments induced by the axial loadings would be additive to those developed by the local wave forces. Finally, since the platform contains joint cans, this analysis used fixed joints.

Loading Results. The initiating failure mode for PC based on the static pile characterization was pile plunging. The force-displacement history for broadside loading is shown in Fig. 5. This curve indicates that platform fails at 0.628 of the reference load pattern or a total base shear of 1700 kips (7.6 MN) at a displacement of about 24 in. (61 cm). From Fig. 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.

Since the results based on the static pile simulation indicated that the foundation was the weak link in the platform, an analysis based on a dynamic pile characterization also was performed.

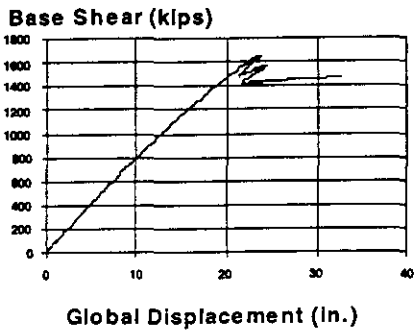

Fig. 5 Platform C force-displacement relationship based on static pile capacities (1 kip = 4.4 kN, 1 in. = 2.5 cm)

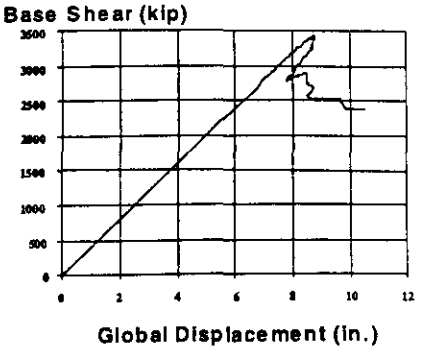

Fig. 6 Platform C force-displacement relationship based on dynamic pile capacities (1 kip = 4.4 kN, 1 in. = 2.5 cm)

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 3440 kips (15 MN). 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 Fig. 6. The peak lateral load capacity is 3500 kips (16 MN) and it is reached at a lateral displacement of about 9 in. (23 cm). Based on the dynamic pile characterization, the lateral load capacity of the platform is about doubled.

Comparison of Analytical and Observed Results. The hurricane hindcast data (Cardone and Cox, 1992) and observed platform performance indicate that PC survived 53 ft (16 m) to 56 ft (17 m) waves during hurricane Andrew. Based on the results from the analyses performed on PC, the total lateral loading associated with these conditions ranged from 1700 kips (7.6 MN) to 1900 kips (8.4 MN). 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 1500 kips (6.7 MN) to 3600 kips (16 MN) (PMB) Engineering, 1994; Digre et al., 1995). Based on the analyses performed during this study, the lower-bound results were obtained when the static pile capacity was utilized (Fig. 5) and the upper bound when the dynamic pile capacity was utilized (Fig. 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) (Figs. 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.

The study of WP1 and WP2 involved the use of three computer programs: 1) StruCad*3D, 2) USFOS, and 3) ULSLEA.

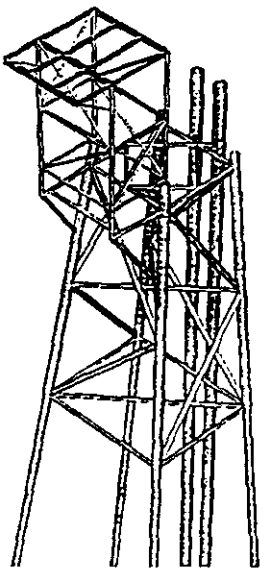

Fig. 7 Wellhead protector 1

StruCad*3D (Zentech, 1993) was used 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) (Bea and Des-Roches, 1993; Bea and Mortazavi, 1996) is a technique to perform simplified analyses of the load-resisting capacities of offshore template structures. 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.

WP1 and WP2 were both located in the South Timbalier area. The two wellhead protectors were designed and installed early in the 1980s by the same firm. The two wellhead protectors were designed according to the same API RP 2A guideline. The slightly older WP1 was located in 52 ft (16 m) of water. The primary horizontal axis of WP 1 was oriented at 315 deg (from true north). WP2 is located in 49 ft (15 m) water about 1500 ft (457 m) to the northeast of WP1. WP2 is oriented with its primary horizontal axis parallel to true north. Both structures were two-bay, four-pile template structures designed to provide limited facilities for 36-in- (91-cm-) dia caisson well risers (Figs. 7 and 8). Both protectors had offset braced helipads and boat landings.

The jacket framing of the two structures was almost identical. WP2 had slightly smaller diameter jacket legs and piles; 28 in. (71 cm) and 24 in. (61 cm), respectively. WP1 had 30-in- (76-cm-) and 26-in-(66-cm-) dia jacket legs and piles, respectively. Diagonal vertical bracing was made up of 18-in-(46-cm) tubulars, while plan bracing was composed of 12.75-in. (33-cm) 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 were located just outside of the structure north end of the jacket and were not tied substantially to the jacket. WP2's caisson was rigidly framed within the interior of the jacket.

The foundations for the two structures were both composed of four piles. The design of these piles was quite different. WP1's piles were 187 ft (57 m) long, 26 in. (66 cm) in diameter, and were comprised of several segments that had different wall thicknesses. At the tip there was a 5-ft (1.5-m) pile driving shoe 0.75 in. (1.9 cm) thick. This section was followed by 100 ft (30 m) of 0.5-in. (1.3-cm) thick pile. Above this segment was the only pile splice below the mudline. Here the wall thickness increased again to 0.75 in. (1.9 cm) for another 10 ft (3 m). The remainder of the pile above the mudline and into the lower bay was 1.125 in. (2.9 cm) thick. WP2's piles were slightly longer (190 ft, 58 m) than those of WP1 to compensate for its smaller diameter of 24 in. (61 cm). Its upper wall thickness was generally larger as well, with a 1.213-in. (3-cm) thickness to withstand the large bending stresses found in the piles near the mudline.

Based on results from soil borings performed at the two locations, the soil conditions were essentially identical and consisted of a deep 172-ft (52-m) thick layer of soft clays overlying a layer of stiff, dense sand. The geotechnical consultant that designed the pile foundations for WP1 and WP2 recommended that the structures' piles should be driven to depth into the sand in order to take advantage of its high compressive bearing capacity.

As for PB and PC, the pile-soil interactions were modeled using API RP 2A guidelines for static lateral and axial pile characteristics and API RP 2A guidelines for dynamic-cyclic loading conditions (API, 1993, 1994).

The results of the study based on StruCad*3D and ULSLEA 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 underdriven by 15 ft (5 m). All of the piles in WP2 had been driven to their design penetrations. The underdriven piles in WP1 resulted in a 15-percent decrease in the pile's tensile capacities.

Wind, wave, and current characteristics were chosen from environmental data provided from the hurricane Andrew hindcast (Cardone and Cox, 1992). As with PB and PC, hydrodynamic coefficients were chosen based on recent test data and engineering judgment. The best-estimate hydrodynamic drag and inertia coefficients for rough cylinders were taken to be 1.2 (Rodenbusch, 1986). A wave kinematics factor equal to 0.88 was used for both the deck and jacket loads. The structure was loaded broadside and end-on.

Wave loads for USFOS were generated using WAJAC (Det Norske Veritas, 1993). The global base shears developed on WP1 and WP2 during the passage of Andrew are summarized in Fig. 9. The results indicate WP1 experienced peak lateral loadings that were about 20 percent larger than those on WP2.

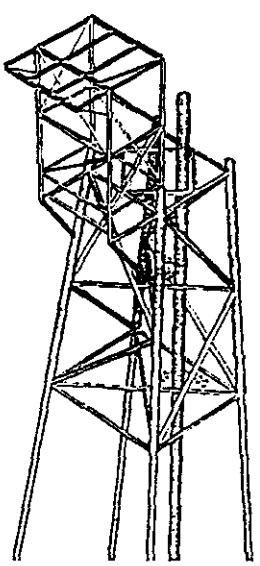

Fig. 8 Wellhead protector 2

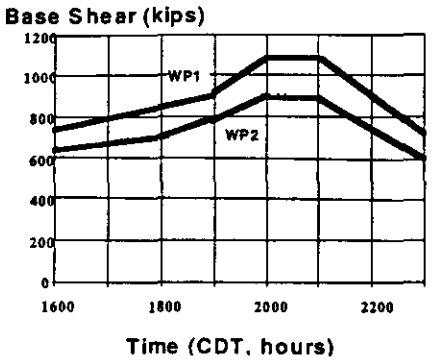

Fig. 9 Time history of base shears on WP 1 and WP 2 during hurricane Andrew (1 kip = 4.4 kN)

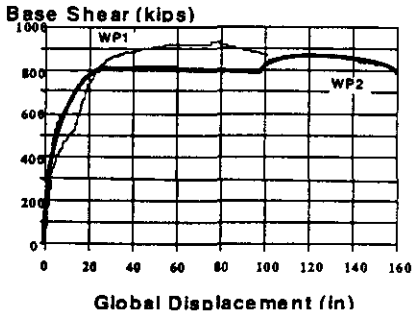

Fig. 10 Load-displacement relationships for WP 1 and WP 2 (1 kip = 4.4 kN, 1 in. = 2.5 cm)

The peak lateral loading on WP1 was 1100 kips, and on WP2 was 850 kips.

Pushover Results. The static pushover results for WP1 and WP2 based on the USFOS results are summarized in Fig. 10. The "double hump" 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 produced plastic hinges in the piles which produced a near mechanism. It was the additional strength and rigidity of the caissons which prevented the structures from forming a mechanism at this point. The added stiffness of the well caissons allowed the full axial capacity of the soils to be exceeded to produce pile pullout. The maximum lateral load capacity of WP1 and WP2 was found to be 910 kips (4.0 MN) and 880 kips (3.9 MN), respectively.

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 was 1.2 and 0.95 for WP1 and WP2, respectively. The analyses indicated 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 caissons were tied into the structures, and the capacities of the underdriven piles. The effects of these differences only became evident when these "details" were determined and their implications integrated into the analyses.

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 eight-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 four-leg platforms that also survived hurricane Andrew. One of these platforms (PC) 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; Digre et al., 1995). 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. PC would not have survived hurricane Andrew based on traditional static pile capacities.

The third four-leg well protector (WP1) was located directly in the path of hurricane Andrew. It collapsed. The fourth nearby, seemingly identical four-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. 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.

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 gratefully 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 Oceanic 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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