

# Analysis and performance of offshore platforms in hurricanes

Ahsan Kareem<sup>†</sup> and Tracy Kijewski<sup>‡</sup>

*NatHaz Modeling Laboratory, Department of Civil Engineering and Geological Sciences,  
University of Notre Dame, Notre Dame, IN 46556-0767, U.S.A.*

Charles E. Smith<sup>††</sup>

*Offshore Mineral Management, Mineral Management Service,  
Technology Assessment and Research Branch, Herndon, VA 22070-4817, U.S.A.*

**Abstract.** Wind effects are critical considerations in the design of topside structures, overall structural systems, or both, depending on the water depth and type of offshore platform. The reliable design of these facilities for oil fields in regions of hostile environment can only be assured through better understanding of the environmental load effects and enhanced response prediction capabilities. This paper summarizes the analysis and performance of offshore platforms under extreme wind loads, including the quantification of wind load effects with focus on wind field characteristics, steady and unsteady loads, gust loading factors, application of wind tunnel tests, and the provisions of the American Petroleum Institute Recommended Practice 2A – Working Stress Design (API RP 2A-WSD) for the construction of offshore structures under the action of wind. A survey of the performance of platforms and satellite structures is provided, and failure mechanisms concerning different damage scenarios during Hurricane Andrew are examined. Guidelines and provisions for improving analysis and design of structures are addressed.

**Key words:** hurricanes; offshore platforms; turbulence; wind loading; building codes; damage; wind speeds; waves.

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## 1. Introduction

The wind effects on platforms can be categorized into two classes, namely local and integral. The local effects concern the design of deck structures, their components and envelopes. The integral loads are responsible for global effects such as overturning moments, base shear, etc. Wind loads account for about 10% of the total environmental loads on traditional shallow water offshore platforms; however, wind loads contribute significantly to the overall design loads for floating production systems and compliant offshore structures in deep water. The wind effects can be expressed in terms of a mean and a fluctuating component. The mean component results from the mean wind force that can be computed from the mean wind

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<sup>†</sup> Professor

<sup>‡</sup> Graduate Student

<sup>††</sup> Program Manager

velocity and an appropriate aerodynamic force coefficient. The fluctuating component results from the buffeting action of wind gusts or wake dynamics. The behavior of compliant structures like tension leg platforms (TLP) in the horizontal plane increases their sensitivity to dynamic effects of winds, which contain a significant level of energy in the low frequency range. This leads to a relative increase in the overall sensitivity and strength requirements of compliant structures to wind loads compared to those of conventional fixed structures.

In the Gulf of Mexico, offshore platforms are often exposed to hurricane wind fields posing a serious threat to the safety and reliability of the deck structures and the overall platform structural system, and may lead to not only loss of operation due to structural damage, but also threaten the environment with possible oil spills. A very large population of gas and oil production platforms fall in the category of traditional shallow water fixed-type jacket platforms. These platforms are more vulnerable to damage of topside structures such as helidecks, towers and cranes, living quarters, and envelopes of enclosed appurtenances. This type of damage was quite prevalent in the aftermath of Hurricane Andrew in the Gulf of Mexico. In addition, the fluctuating local load effects become significant as the fatigue action induces premature failure of fasteners and components.

Hurricane Andrew was not a very strong hurricane as it moved through the Gulf of Mexico, but the damage inflicted by wind was very significant. This suggests that either the platforms that experienced damage were not adequately designed for wind/waves, or over the years of operation, fatigue and other causes of degradation had reduced their resistance to loads.

To assess the performance of existing structures under future hurricanes and to develop retrofiting and mitigative measures, understanding of the wind field and its effects on the offshore built environment is essential.

## **2. Loading analysis**

The following section discusses the wind loads, both steady and unsteady, acting on offshore structures. Essential to the quantification of these loads is an understanding of the wind field characteristics over the ocean, which is addressed in subsection 2.1. Finally, an overview of the wind loading provisions in API RP 2A – WSD, the design standard for the offshore community, is provided.

### *2.1. Wind field characteristics*

There is a limited amount of data available from wind measurements taken over the ocean, and published results are uncertain, especially when extrapolated to extreme design conditions. The problem is essentially due to the practical difficulty of taking measurements and is compounded by the variable nature of the sea surface. Data over the ocean suggests that the mean velocity profile and turbulence structure are similar to those of the smooth terrain despite the presence of large waves. The only difference is the continuous translation and deformation of the sea surface. The large waves move as fast as strong winds; therefore, the contribution of waves to the surface roughness is small as compared to the small waves which tend to break. This feature explains the relative low resistance of the ocean surface despite the presence of large waves. Thus, the wind flow field and its characteristics are not influenced significantly by the exact form of the sea

surface. Rather, these are sensitive to the energy loss and the rate of the momentum transfer due to the surface friction. This clearly suggests that the relationships developed for on-land wind characteristics may be extended to the flows over the ocean. However, some studies have shown dependence on wind speed, fetch, water depth and the evolutionary stage of waves (Donlean 1982, Forristall 1985, Brown and Swail 1991, Anctil and Donlean 1996).

Regarding the unsteady wind field, surface roughness introduces the surface shear stresses and turbulent fluctuations in the boundary layer flow. Besides mechanically generated turbulence, thermal effects can enhance (unstable conditions) or retard (stable condition) the transfer of turbulent energy. It is generally noted that in moderate to strong winds mechanically produced turbulence from surface roughness dominates the convective turbulence. However, observations have been recently made in the North Sea tend to demonstrate otherwise. It is also important to investigate the hurricane wind field model to parameterize the evolution of localized convective cells that are characterized by excessive rain bands and strong gusts. These intense drafts may facilitate stretching of vortices induced by winds of different speeds spiralling towards the low pressure region. The stretching process may enhance the rotational speed, which, in combination with the winds bounding these vortices, may lead to very high localized intensification of wind speeds (Gore 1993). Some strongly dispute this localized spiral-induced intensification of winds.

The wind spectrum over the ocean lacks a universal description, and measurements exhibit a wide scatter (Kareem 1985, 1998). The presence of large scatter, in addition to the statistical variations, can be attributed to air-sea temperature difference and partially result from the nonlinear interactions of different frequency components near the sea surface which redistribute energy at different frequencies. An accurate description of the wind spectrum is important for establishing wind gust factors, and so a new spectral description has been proposed (Kareem 1985, Chakrabarti 1990). The spectral description for wind over the water is given by :

$$\frac{fS_u(f)}{u_s^2} = \frac{An}{(1+Bn)^{5/3}} \quad (1)$$

where  $S_u(f)$  is the single-sided spectral density,  $f$  is the frequency in Hertz,  $u_s$  is the friction velocity,  $n$  is the reduced frequency defined as  $n = fz/\bar{U}$ , where  $\bar{U}$  is the mean velocity at height  $z$ , and  $A$  and  $B$  are determined from environmental parameters. For example, if  $\bar{U}(10) = 20$  m/s, then  $A$  and  $B$  take on values of 335 and 71. Subsequent analysis has shown that the proposed model adequately describes the full-scale data from the Gulf of Mexico, North Sea, Pacific and Atlantic Oceans. Additional improvements to incorporate the convective source of low frequency contributions is currently being studied.

The spatial correlation of the wind field is essential for describing wind loads that adequately reflect the true correlation structure of the wind field. It is noted that the coherence function generally used for tall land-based structures inadequately describes the spatial structure of the wind field over the ocean for spatial separations that may exceed the turbulence length scale and is influenced by the closeness of the ocean surface. Modifications have been proposed that include a number of concerns. Wind tunnel experiments have been conducted in which the flow over smooth terrains was simulated and the measured data was reduced to provide coherence for a wide range of separation distances (Kareem 1992). These measurements and some additional data available from other wind tunnel studies and full-scale data over smooth and rolling terrains

were utilized to establish the coherence model. A proposed expression for the coherence of ocean-based structures is (Kareem 1987)

$$\text{coh}(\Delta y, \Delta z, f) = \text{coh}(\Delta y, f) \text{coh}(\Delta z, f) \quad (2)$$

$$\text{where } \text{coh}(\Delta z, f) = \exp \sqrt{-\left(a_z \frac{\Delta z}{l}\right)^2 + \left(\frac{f}{U_{12}}\right)^2 \left(b_z \Delta z + \frac{c_z (\Delta z)^2}{Z_{12}}\right)^2} \quad (3)$$

$\Delta y$  and  $\Delta z$  are the separation distances in the  $y$  and  $z$  directions,  $l$  is the length scale of turbulence,  $Z_{12}$  is the average height of the locations,  $U_{12}$  is the average velocity between the two locations, and the coefficients are defined as:  $a_z=0.2$ ,  $b_z=2$  and  $c_z=10$ . To determine  $\text{coh}(\Delta y, f)$  replace  $\Delta z$  with  $\Delta y$  and  $a_z$ ,  $b_z$ , and  $c_z$  with  $a_y=1$ ,  $b_y=2.5$ , and  $c_y=16$ , respectively, in Eq. (3).

It is noted that the coherence given by this model, for both horizontal and vertical separations, is lower than that given by the expression generally used over land and the model based on isotropic turbulence. For wind-induced surge response, the present model will tend to give lower estimates in comparison with those based on the coherence function that is typically employed; however, for yaw response, this will result in a higher response. This may have significant implications for the design of deep water platforms with asymmetric deck geometry and layout of deck structures.

## 2.2. Wind effects

As mentioned previously, wind loads on structures may be described in terms of steady and unsteady components, which are treated separately in the following subsections. A discussion of steady loads and their estimation via aerodynamic force coefficients is presented, along with a discussion of unsteady loads, quantified using the gust loading factor. A critical evaluation of these approaches is also provided.

### 2.2.1. Steady loads

The steady wind loads on a structure are expressed in terms of the wind velocity and aerodynamic force coefficients. The aerodynamic force coefficients are generally derived from code recommended values (API 1993, Det Norske Veritas 1982, API 1995) or alternatively can be obtained from scale models exposed to simulated flow in a wind tunnel (Kareem *et al.* 1987, Williamsen 1992).

The synthesized aerodynamic force coefficients for platforms based on a projected area approach utilizing force coefficients for individual structural shapes are derived without specific consideration of complex shielding, lifting forces and other three dimensional flow effects. The problem of accurate prediction is complicated by the presence of complex three dimensional vortical flow structures that originate from the flow separation regions and subsequently engulf the platform deck structure partially or even completely (Kareem *et al.* 1987).

The codes and standards typically fail to predict overturning moments caused by lifting

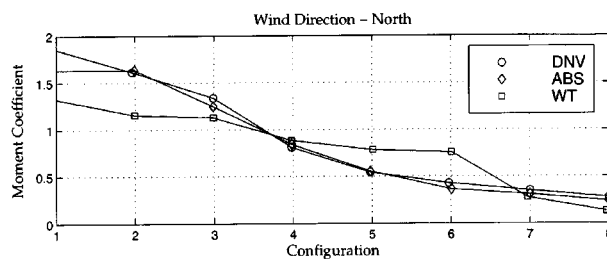


Fig. 1 Comparison of predicted and measured drag-induced moment coefficients – wind approaching from  $0^\circ$

surfaces such as a platform deck or a helideck. Many platforms in Hurricane Andrew experienced damage to their decks. This study will examine the overturning moments induced by various deck configurations. The influence of local wave profile on the upflow angle, local intensification of flow and the changes in the angle of flow can be significant. The influence of a lifting surface is demonstrated in Fig. 1 in which the drag induced moment (overturning moment) is plotted for different platform configurations of a TLP utilizing DnV (DNV), American Bureau of Shipping (ABS) and wind tunnel results (WT) (Kareem *et al.* 1987). A total of eight platform configurations were investigated in this study that included all the ancillary structures to the case where every deck component was removed. The results in Fig. 1 suggest that the estimated coefficients of drag-induced moment were lower than the corresponding wind tunnel measurements for configurations in which the drilling derricks had been removed but the helideck and the living quarters were still in place. In these configurations the flow-induced aerodynamic lift contributes to the pitching moment. As discussed earlier, the projected area approach fails to include this contribution which leads to underprediction of the drag-induced moment. In configurations 7 and 8, the removal of living quarters and helideck eliminated the dominant source of flow induced lift force that was responsible for the departure of predicted values from measured coefficients (Kareem *et al.* 1987).

### 2.2.2. Unsteady loads

The unsteady aerodynamic loads are quantified in terms of “Gust Loading Factors” (GLF) which embody the concept of equivalent static loading based on dynamic effects. In its most basic form, the GLF may only include the dynamic effects of wind fluctuations, whereas, for more flexible structures, the dynamics of the structure needs to be incorporated to account for the amplification that results from structural oscillations. A random vibration based approach is utilized that begins with the space-time structure of the wind field and following strip and quasi-steady theories leads to dynamic loads. Small eddies in the turbulent flow impinge successively, whereas larger eddies are well correlated and act instantaneously. Overall dynamic effects result from the synthesis of space-time local averages of random pressure fields that accounts for lack of spatial and temporal fluctuations. The loading spectrum is convolved with the transfer function representing the dynamic characteristics of the structure, and subsequent analysis of extremes leads to quantification of the complete dynamic effects. A normalization with the overall static loads results in the desired GLF (Gurley and Kareem 1993). Closed-form expressions are

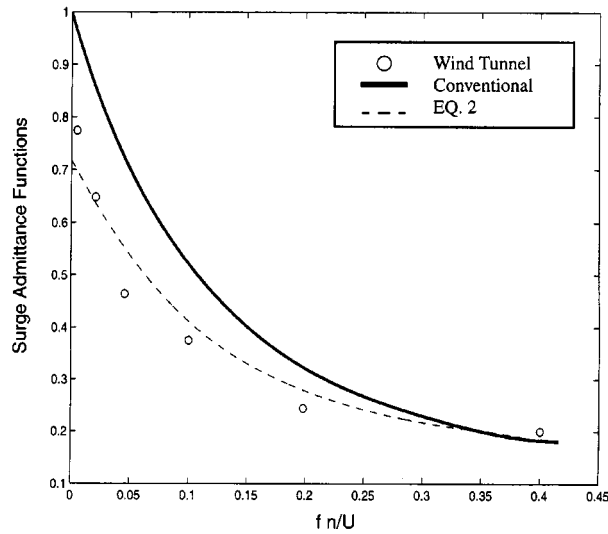


Fig. 2 Surge admittance function

available in ASCE7-95 for land-based structures (ASCE 1995, Kijewski and Kareem 1998). In the case of offshore platforms, the GLF can be equivalently derived following the procedure outlined by Gurley and Kareem (1993).

The validity of quasi-steady and strip theories has been examined through measured unsteady aerodynamic loads on scale models of TLPs (Gurley and Kareem 1993). Aerodynamic admittance functions that represent the transfer function between the wind fluctuations and the force fluctuations were computed and compared to experimentally determined values. The results, shown in Fig. 2, are in good agreement with the coherence function that takes into account the length scale of turbulence as compared with conventional coherence, employed for tall buildings and towers, that tends to overestimate. Similar observations have been made for offshore guyed tower analysis (Vickery and Pike 1985). The trend is reversed for yaw loading, which results from asymmetry in flow, where conventional coherence underestimates due to higher correlation.

Typically the GLF formulation ignores the nonlinear velocity term in the loading formulation. The presence of these terms becomes more significant for higher turbulence levels. Furthermore, the nonlinearity in the loading terms precludes the use of the extreme value analysis based on the assumption of the parent process being Gaussian. In a recent study by the first author, an equivalent quadratization approach based on Volterra series representation is utilized to obtain the higher-order statistics of the platform response in terms of skewness and kurtosis (Zhao and Kareem 1994). A subsequent Hermite moment-based transformation facilitates analysis of the extremes which leads to GLF. Alternatively, a Gram-Charlier series representation is also utilized in the analysis to describe the response that departs from Gaussian. It is noted that the inclusion of nonlinearity enhances the value of GLF and Gram-Charlier description fails to represent the extremes for increasing level of nonlinearity. The GLFs have been further examined in the light of uncertainty associated with numerous parameters and variables that are involved in their formulation (Gurley and Kareem 1993). The propagation of uncertainty is based on a Monte Carlo formulation. This analysis may be further carried out to accomplish a reliability analysis by

examination of the limit state function designed to determine conditions leading to a failure mode. The failure mode more specifically may be exceeding performance levels necessary for a particular intended operational or safety function.

### 2.3. Treatment of wind effects in API RP 2A

Before discussing the damage experienced by offshore platforms during Hurricane Andrew, an overview of the treatment of wind effects in API RP 2A-WSD (API 1993) section 2.3.2. is provided. While API 4F : Specifications for Drilling and Well Servicing Structures (API 1995) are also available, these specifications will not be discussed herein, since their focus is only on pipe stacks and derricks on the platforms. The discussion of wind effects in standards for offshore structures that follows is limited to API RP 2A-WSD, which details procedures for determining the wind loads acting on portions of the platform above water as well as equipment, deck houses, and other structures which are located on the platform, noting the significance of wind loads in deeper waters and for compliant designs.

The provisions permit the classification of wind in terms of gusts averaged over less than one minute or sustained winds which are averaged over a minute or longer; however, the wind speeds should be adjusted to a reference height,  $z_{ref}$ , of 10 meters above mean water level and averaged over a period of typically one hour. Section 2.3.2. provides expressions for determining the averaged wind speed for any elevation,  $z$ , based on standard profiles and gust factors. The profile for mean hourly wind speed is given by :

$$V(1\text{ hr}, z) = V(1\text{ hr}, z_{ref}) \left( \frac{z}{z_{ref}} \right)^{0.125} \quad (4)$$

A gust factor, based on extreme value excursion statistics, representing the most probable or mean extreme wind velocity value or its resulting load effect, is used for determining equivalent static wind loadings in most codes and standards. As noted in the previous section, this approach relies on random vibrations theory to translate the dynamic amplification of loading, caused by turbulence and the dynamic sensitivity of the structure, into an equivalent static loading.

The present gust loading factors for designing offshore facilities according to API RP 2A are based on the American National Standards Institute (ANSI) Minimum Design Loads for Buildings and Other Structures (ANSI 1982), which was prepared for land-based structures. A critical evaluation of the gust loading factors currently used by API RP 2A is an essential prerequisite for safety evaluation of deck structures and the overall wind effects. The gust factors must take into account the wind field characteristics over the ocean, provision of air mixed with water spray, inclusion of the square of wind velocity term (dropped for land-based structures), and the effects of localized convective cells in the hurricane wind field that are characterized by excessive rain bands and gusts.

In API RP 2A, the ratio of the wind velocity averaged over time  $t$  at elevation  $z$  to the mean hourly wind at that same elevation, which defines the gust factor  $G(t, z)$ , is described by :

$$G(t, z) = 1 + g(t)I(z) \quad (5)$$

where :

- $t$  is the gust duration [sec]
- $g(t)$  is a peak factor determined by  $g(t) = 3.0 + \ln[(3/t)^{0.6}]$  for  $t \leq 60$  sec
- $I(z)$  is the turbulence intensity, defined as the standard deviation of the wind speed normalized by the mean hourly wind speed and is given by

$$I(z) = 0.15 \left( \frac{z}{z_{ref}} \right)^{-0.125} \quad \text{for } z \leq z_s$$

$$I(z) = 0.15 \left( \frac{z}{z_{ref}} \right)^{-0.275} \quad \text{for } z > z_s \quad (6)$$

where  $z_s$  is the thickness of the surface layer, defined as 20 m.

From the wind pressures introduced to the superstructures, equivalent forces,  $F$ , may be determined by :

$$F = \left( \frac{w}{2g} \right) V^2 C_s A \quad (7)$$

where  $w$  is the density of air [lbf/ft<sup>3</sup>],  $g$  is acceleration due to gravity [ft/s<sup>2</sup>],  $V$  is the wind speed [ft/s],  $A$  is the object area [ft<sup>2</sup>], and  $C_s$  is the shape coefficient, which, in the absence of other data, is given in the standard for standard shapes, assuming a perpendicular wind approach angle with respect to the projected area.

Since wind gusts have three dimensional spatial scales related to their durations, appropriate gusts must be selected based on the objective of the design. For example, the 3 second gusts, being coherent over shorter distances, affect smaller elements more significantly and are thus typically used for determining the maximum static load on individual members. Appropriately, 5 second gusts are suggested for maximum total loads on structures whose horizontal dimension is less than 164 ft (50 m) and 15 second gusts should thus be used for larger structures or their sections. The one minute sustained wind is recommended for the total static superstructure wind loads associated with maximum wave forces for structures that respond dynamically to wind but do not require full dynamic treatment, while structures without significant dynamic response can be analyzed using mean hourly winds. API RP2 also suggests that, for turbulent fluctuations, the flow field may be considered fully coherent over the entire superstructure, which may lead to overestimation of loading in the surge direction, but underestimates for yaw.

In situations where the wind field contains significant energy content near the resonant frequencies of the platform, as is often the case with compliant structures like guyed towers and tension leg platforms in deep water, a full dynamic analysis is required. To establish if this is indeed the case, a frequency analysis of the incoming wind is required. Unfortunately, as noted earlier, due to the large variability in wind data, there is no standard spectral shape; however, unless available data indicates otherwise, the wind spectra can be generated by :



$$\frac{fS(f)}{\sigma(z)^2} = \frac{f/f_p}{[1 + 1.5(f/f_p)]^{5/3}} \quad (8)$$

where  $S(f)$  is the power spectral density at elevation  $z$  and  $\sigma(z)$  is the standard deviation of the wind speed which is equivalent to  $I(z)V(1\text{ hr}, z)$ . This spectra is similar to that given in Chakrabarti (1990), but frequency value  $f_p$  is suggested to be varied within a prescribed range for wind sensitive platforms to account for the large variability in data, according to :

$$0.01 \leq \frac{f_p z}{V(1\text{ hr}, z)} \leq 0.10 \quad (9)$$

Finally, recognizing the significance of envelope rupture, a common failure mode witnessed in the wake of Hurricane Andrew for both onshore and offshore structures, the standard also gives some consideration to local wind force effects and internal pressures in section 2.3.2(d). For further evaluation of localized pressure concentrations and the effect of internal pressures, API RP 2A suggests reference to section 6 of ANSI A58.1-82 : Minimum Design Loads for Buildings and Other Structures (presently known as ASCE-7) (ANSI 1982, ASCE 1995).

However, the adequacy of the current API RP 2A for the topside structures needs to be assessed, since a straightforward application of the ANSI Standard for different conditions is not always appropriate. The approach flow conditions for land-based and platform topside structures and components are quite different. The flow separating from the deck's leading edge introduces wind in some regions much higher than the far-field winds. These features may provide some answers to the local failures of topside structures either due to metal rupture at the connection or due to local pressure exceeding design values witnessed during Hurricane Andrew.

### 3. Performance analysis

In the following section, a brief overview of the wind speeds observed in Hurricane Andrew is presented, followed by a summary of the performance of platforms and satellites during the event (Kareem and Smith 1993) along with a detailed description of some specific instances of damage.

#### 3.1. Wind field

For a more complete understanding of the wind field generated by Hurricane Andrew, hindcast studies have been consulted, such as the analysis performed by Cardone and Cox (1992) which estimated maximum wind speeds of 50 m/s due north of the storm center. Note that the wind speeds provided by the study were defined as the effective over-water mean hourly winds 20 meters above sea level, thus the 10 m wind speeds which are used in the API RP 2A standard will be approximately 8% lower. Due south, however, the wind speeds dropped to 35 m/s, indicative of the asymmetry typically observed in Gulf Hurricanes. Directly along the hurricane path, as it moved from the southwest coast of Florida into the Gulf, wind speeds were approximately 40 m/s; however, as the hurricane approached the Louisiana Coast, passing through South Timbalier, Ship Shoal, and South Pelto, wind speeds were in the vicinity

of 45-50 m/s, exceeding 50 m/s over only a small area of the southwest Mississippi Delta. Upon verification of this hindcast study with the limited actual measured wind speeds during the event, it was observed that hindcast winds were generally conservative measures of the actual values. Additional information on the wind speeds observed in Hurricane Andrew are also discussed in NOAA reports (NOAA 1992).

### 3.2. Hurricane Andrew performance analysis

On August 25, 1992, Hurricane Andrew began its rampage through a large population of offshore oil and gas facilities in the central Gulf of Mexico, leaving a 50 km path of destruction in its wake. The hurricane passed over areas such as Ship Shoal, South Timbalier, South Pelto and Grand Isle, in the relatively shallow waters of the Mississippi Delta (Fig. 3). This region is known for giving birth to Gulf of Mexico oil exploration, with some of the platforms dating back to the late 50's and 60's. The petroleum exploration and production infrastructure in the direct path of the hurricane were heavily damaged and some completely destroyed. Most of the damage was experienced by aged and simple structures that were designed before recent changes that have made offshore installations more robust and more resistant to storms like Andrew. In particular, single well caissons, also termed satellites, which were constructed at a time when design codes were evolving, often through trial and error, suffered significant damage. Those constructed before 1987, when standards for installation of such structures were developed, were hit hardest. A similar fate befell platforms designed by older criteria. Traditionally, a 25 year storm criteria was used in design of platforms rather than today's 100 year storm criteria which was first established in 1976. As a result, when the 59 foot waves of Hurricane Andrew crashed into these older structures, they had drastic impacts. While

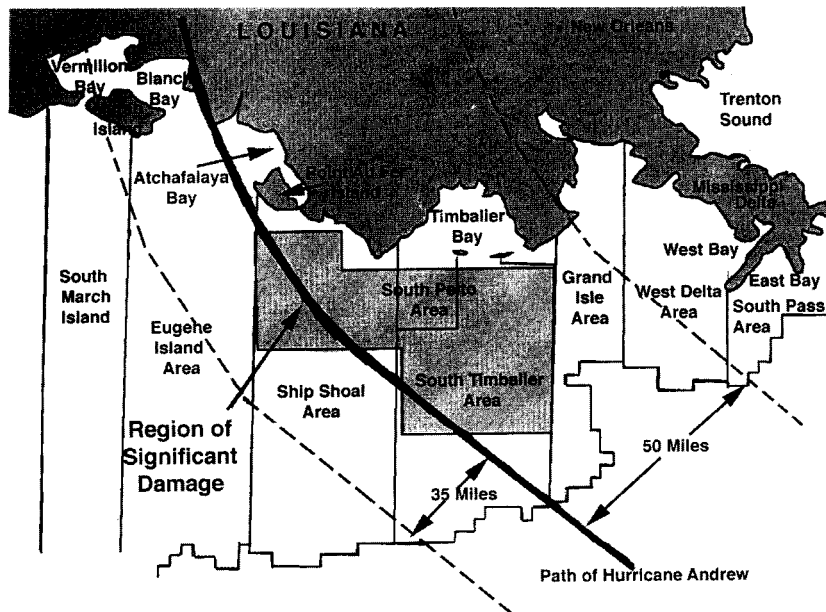


Fig. 3 Path of Hurricane Andrew and region of heavily damaged structures

these waves are just under today's 100 year storm condition, they exceeded the wave height designed for by the 25 year storm criteria, resulting in forces on the older platforms which were nearly double those anticipated (Imm *et al.* 1994). Surprisingly, however, many older structures, greater than 40 years old, did survive the storm with only minimal damage, such as those to handrails and gratings (MMS 1992).

For the most part, the structures which suffered the most damage and even failed were in less than 30 m of water, with the majority failing within 40 km of the shore. The location of the most heavily damaged and even collapsed platforms, shown in Fig. 3, illustrates this observation. In fact, 68% of platforms and 98% of satellites which failed were in less than 30 m of water. This statistic is alarmingly high due to the fact that many of the older structures are in shallow water. The high percentage of satellites failing in less than 30 m of water is also biased by the fact that satellites are more economical to construct in shallow water and therefore more of them are located at such depths. There has been some speculation as to other reasons why failure was so significant in shallow water. One theory suggests that as large waves moved into shallower waters, they grew steeper and broke against the structures in their path. The tremendous force generated by the waves, coupled with the fact that they were high enough to reach the decks, may account for the large number of failures in 7~23 m of water.

Unfortunately, the hurricane did not expose modern deepwater platforms equipped with sophisticated instrumentation systems, which has left the designers of these structures without any feedback as to how these large scale systems performed during the storms. For example, Andrew passed some 80 km north of Conco's TLP at the Jolliet field and missed Exxon's Lena guyed tower, where winds of about 32 m/s were recorded (Cardone and Cox 1992), by a similar distance. (Wind speeds at the Bullwinkle Platform were observed to be approximately 27 m/s (Cardone and Cox 1992)). These technologically advanced modern drilling and production platforms that are designed to withstand environmental loads introduced by a storm like Andrew are expected to perform well in future hurricanes.

While Hurricane Andrew dealt a severe blow to the region, it resulted in very little environmental pollution. While it is estimated that there were 396 locations of pipeline damage, the shutdown of lines prior to the hurricane's arrival prevented serious environmental damage. In fact, it was predicted that less than 2500 barrels of oil were spilled, 2000 barrels of which were spilled at just one location (Botelho *et al.* 1994b). This spill was attributed to a massive semi-submersible drilling rig that had been stacked for several years in South Pelto Block 7 became adrift. The rig was tossed about in Andrew's waves as it dragged its four, 30,000 lb anchors along the ocean floor. One of these anchors struck a pipeline. Despite being coated in 12.86 cm of concrete, the pipe suffered severe deformation and upon its restart 2 days later, a 10 cm gash along the longitudinal weld seam erupted. The oil loss was contained to 2000 bbl (Lewis *et al.* 1993). While the U.S. Coast Guard reported 75 oil slicks, only 7 were significant enough to be considered spills, and even these occurrences had minimal environmental impact (MMS 1992). In addition, two fires were also reported (Fig. 4).

Of the estimated 3,850 offshore structures throughout the Gulf of Mexico's outer continental shelf, some 2,000 were exposed to Andrew's hurricane-force winds. Of those structures exposed, only 243 sustained damage, with 112 of those suffering structural damage (Koen 1992). It should be reiterated that many of those structures damaged were older structures. Specifically, 800 of the 2000 exposed structures were installed prior to 1976, when the tougher 100 year

storm criteria was imposed (Offshore Engineering 1992). In fact, all the structures that were totally lost were of the pre-1976 design criteria (MMS 1992). It is also interesting to note that older structures built before the 1960's may have been easily toppled due to fatigue received as a result of exposure to an earlier hurricane, Betsy, in 1965 (Lewis *et al.* 1993). Despite the general damage tabulated below, it is some consolation to note that there were no fatalities in the offshore community as a result of Andrew.

The damage can be generally described by several categories as shown in Table 1, along with an illustration of the particular damage scenario.

The major damage to area facilities is briefly summarized by Table 2. Most of the minor damage encompassed roofs, siding, boat docks, ladders, stairs, walkways, and heliports. Other significant damage included toppled microwave towers, toppled satellite dishes and generators lost due to flooding (Tubb 1992). Also observed was the consistent failure of multiple K-joints in 4 of the failed platforms and 2 damaged platforms (Puskar *et al.* 1994). The damage is attributed to both actions of wind and waves, in which winds were primarily responsible for topside deck

Table 1 Types of damage observed during Hurricane Andrew

Damage Scenarios	Examples
Wind/wave damage to deck appurtenances, satellite dishes, helidecks, etc.	Fig. 5
Heavy damage to platform structural systems/Toppled platforms/Collapsed Platforms	Fig. 6
Deck structure damage : walkways, railings and stairs damaged or missing	Fig. 7
Heavy equipment toppled/moved	Fig. 8
Platforms leaning	Fig. 9
Single well satellites leaning/toppled	Figs. 10~12
Living quarters or other enclosed structures damaged	Fig. 13

Table 2 Major damage witnessed during Hurricane Andrew

Damage Scenario	Number of Occurrences
Platforms toppled	45
Platforms bent over	89
Platform suffered irreparable structural damage	43
Platform suffered significant, but repairable damage	100
Mobile drilling units set adrift	5

Table 3 Damage to satellites or single well caissons

Damage Scenario	Number of Occurrences
Toppled Jackets	41
Jackets leaning < 5°	43
Jackets leaning 6~10°	13
Jackets leaning 11~15°	6
Jackets leaning 20~25°	5
Jackets leaning 30~35°	7
Jackets leaning > 45°	5

Note : All angles measured with respect to the vertical

structure damage and contributing towards toppling, leaning and collapse of structures.

Other, more detailed statistics based on tallies from a November 1992 hurricane damage report, summarized in Tables 3 and 4, reveal the significance of the damage.

The damage tally in Table 5 was provided by the Mineral Management Service (MMS) and details both significant and minor damage observed (Note that the terminology used to categorize the damage is that of the MMS).

Table 4 Actual platform damage

Damage Scenario	Number of Occurrences
Platforms and/or rigs leaning	4
Barges leaning	2
Platforms toppled	13
Platforms lost	1
Lost production decks	1

Table 5 MMS damage tally

Damage Scenario	Number of Occurrences
Collapsed/Toppled Platforms	13
Sites damaged beyond repair	2
Instances of "heavy" damage	20
Instances of "minor" damage	12
Single well satellites toppled	12
Toppled wells	2
Damaged satellite dishes	2
Damaged/missing decks	9
Damaged/missing walkways	1
Heavy equipment toppled	1
Damaged/missing stairwell	3
Pipeline damage	4
Wave damage	1
Heliport damage	2
General damage to/ leaning of platform	9
Jacket toppled	1
Generator damage	2
Missing satellite	1
Missing platform	3
Leaning Jacket	4
Damaged living quarters	1
Toppled satellite	2
Towers falling onto other structures	2
Damage to navigational aids	1
Flare boom damaged	1
Wind damage to appurtenances	1
Breakaway of platform	1
Oil slick	3
Fire	3

Table 6 Botelho, Ullmann, Chancellor &amp; Versowsky damage assessment

Damage Scenario	Number of Occurrences
Significant damage to major platforms	36
Completely toppled	10
Leaning/significant topside damage	26
Significant damage to major satellites	165
Completely toppled	25
Leaning 1~5°	77
Leaning 5~45°	43
Barrels of oil spilled	2500

Another damage assessment by Botelho, Ullmann, Chancellor and Versowsky (1994b), is listed in Table 6.

This study was also able to document several pieces of evidence to support topside wave



Fig. 4 Remaining part of a platform under fire

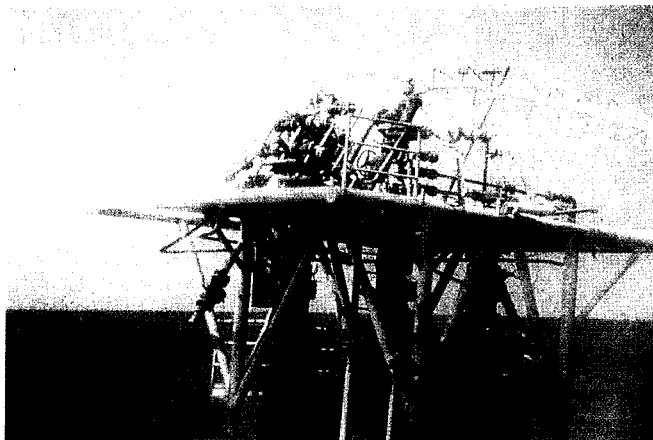


Fig. 5 Significant damage to deck appurtenances



Fig. 6 An example of a platform with missing deck structure

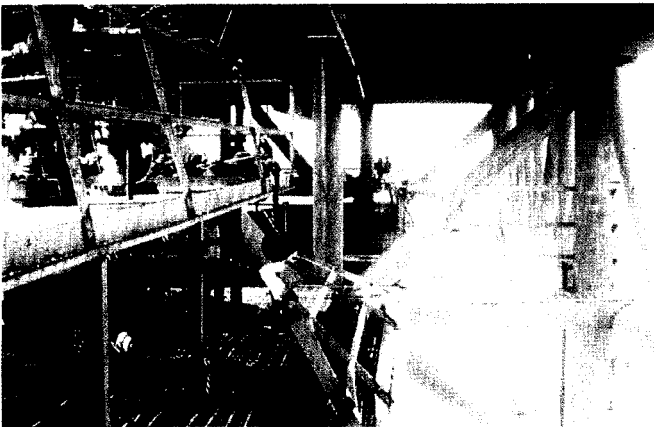


Fig. 7 Damaged railings on platform deck

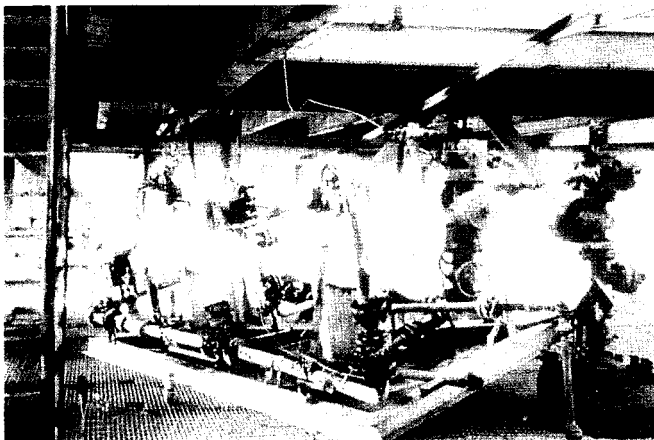


Fig. 8 Heavy equipment displaced on platform deck

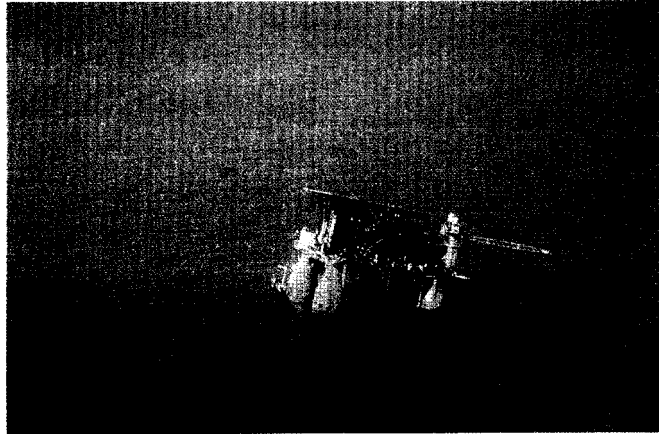


Fig. 9 Platform leaning 15~20° from vertical

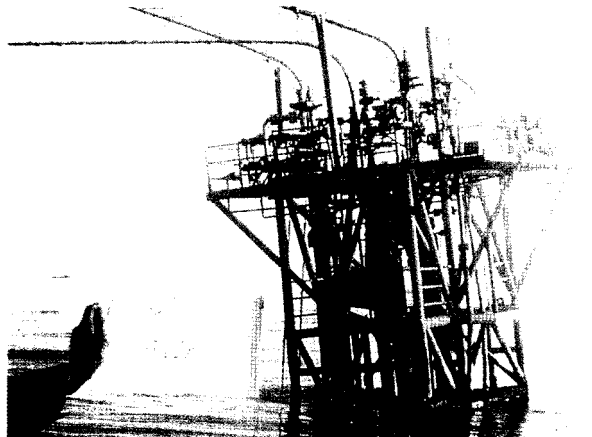


Fig. 10 Single-well caisson leaning 6~10°

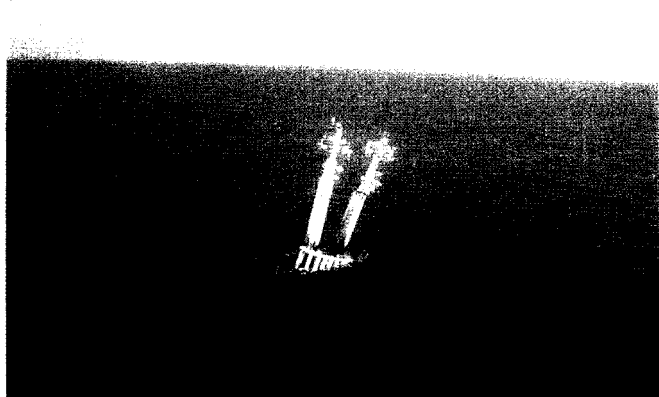


Fig. 11 Toppled satellite



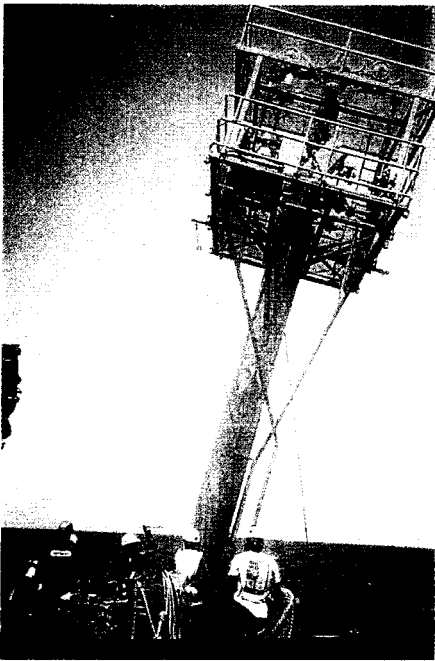


Fig. 12 Example of leaning caisson

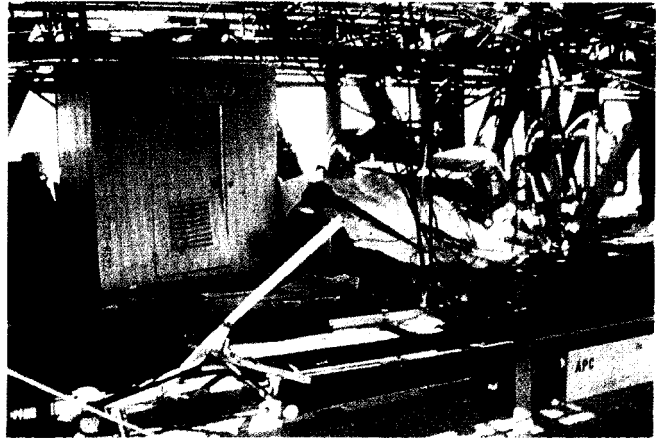


Fig. 13 Damage to deck structures including a small utility enclosure

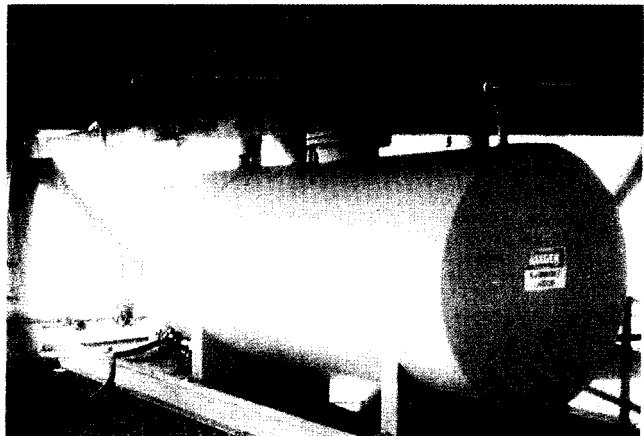


Fig. 14 Large tank shifted out of position on deck

force damage : (1) bent handrails, stairways, and heavy deck support beams on the cellar deck level, (2) severely bent swing rope support beams, and (3) steel framed well head signs installed on opposite sides of each well head and bent in line with the wave direction. Other less conclusive evidence, such as the movement or upending of heavy equipment (Figs. 14 & 15) and deck hatches, also supports theories of wave damage. The team used post-survey calculations to illustrate that some of the damage found, like the bending of swing rope supports, could not have been caused by the wind. Of the three fields they surveyed in the South Timbalier area, deck damage was limited to the loss of some handrails and several small utility buildings and equipment baskets. In addition, hatches were uplifted from their resting places and large signs on the deck were in various states of damage. The team found that the cellar decks typically suffered local damage. The damage consisted of bent stairways and handrails (Fig. 16), lost gratings, and pneumatic control panels and tubing displaced from their normal locations. In many instances, the swing-rope support beams located at the main deck or cellar deck were severely bent. Small signs

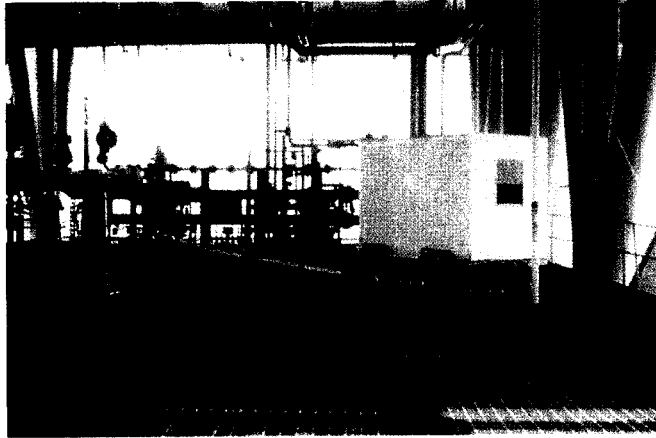


Fig. 15 Significant movement of heavy equipment across platform deck

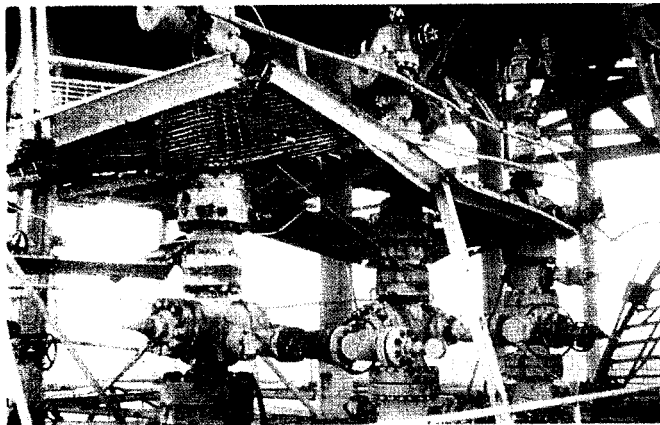


Fig. 16 Heavily damaged walkway and railings

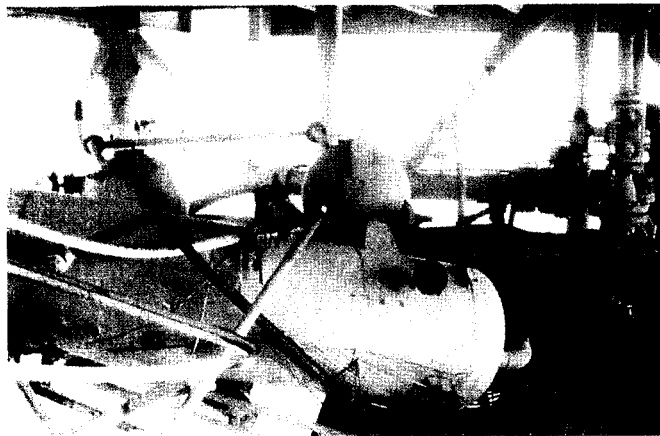


Fig. 17 Damage to deck appurtenances



Fig. 18 Damage to deck railing and ladder

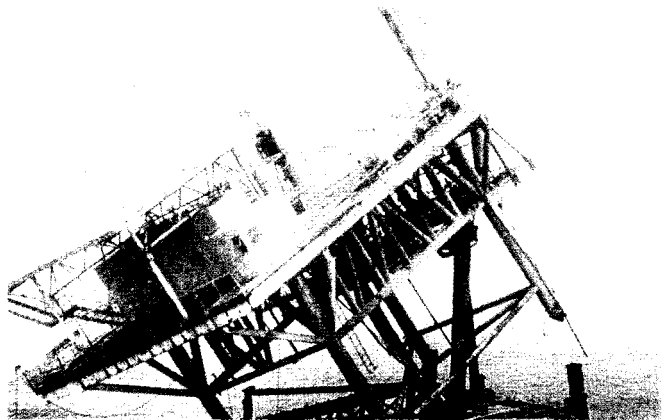


Fig. 19 Partially uplifted deck of damaged platform



Fig. 20 Heavily damaged platform with deck module

were found to be consistently bent in the direction of the waves. In addition, heavy equipment was found to be transported substantial distances across the deck. The worst damage was observed in the subcellar decks, typically located below 9 m in elevation, in which handrails, kickplates, and deck grating was severely bent or lost. On the Plane "A" level of the structures, boat landings and stairways to the cellar deck were severely damaged or even lost. Handrails and gratings were also stripped from the structures (Botelho *et al.* 1994b).

It is especially alarming to consider the total failure of some structures. About 10 major platforms and 25 satellite platforms, mostly caissons, were toppled during the storm causing damage to 26 other platforms and 140 other satellite platforms. Among the structures which failed completely was the ST130 "A" Platform, the first platform installed in 140 foot water in the ST131 field of South Timbalier area of the Gulf of Mexico. The platform, designed in 1957 and installed in 1958, was an 8-pile self-contained drilling and production platform designed for the 25 year storm. Following Andrew, inspections revealed that several members had failed on the bottom 2 bays, the remainder of the platform, above the 2 bottom bays, survived, resting to the northwest of its original location, which was the direction of the most forceful waves, mud-



Fig. 21 Typical structural envelope damage analogous to that witnessed on land



Fig. 22 Damage to corrugated enclosure

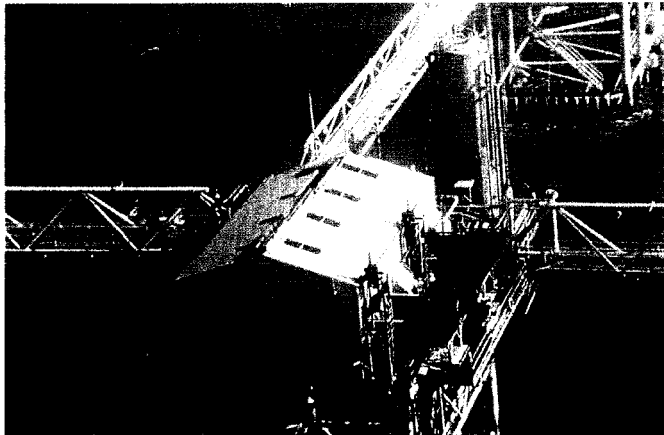


Fig. 23 Aerial view of missing ridge sections of corrugated roofing

line elevation framing remained in-place after the collapse of the platform. There was no pile plunging or pull-out and deformation of the legs at severance points occurred which indicates that the legs were bent about an axis parallel to the platform short axis (Botelho *et al.* 1994a).

A specific outline of the damage to an older structure is offered in a study by Imm, O'Connor and Light (Imm *et al.* 1994). The study surveyed the damage to South Timbalier 161 A Platform constructed in 1964. The 8 leg structure featured both vertical diagonal bracing and K-bracing. The main damage, ranging from paint cracking up to permanent deformations, was to the K-joints. Clearly the K-joints had suffered a high degree of strain. Other specific damage consisted of a horizontal member punching the jacket leg wall inwards until it touched the inner pile. This damage occurred on two of the four K-joint rows. In addition, the horizontal member partially pulled away from the jacket leg in two instances, resulting in fracture.

Other forms of damage included the detachment of pipelines which were found at various distances from their original location. For instance, pipelines which were broken loose at the

breakaway connection at one platform were later found 122~152 m away. In fact, at one site, 9 flowlines were damaged with one being broken loose at the riser and found 1220 m from that location. While such underwater damage to pipelines and structural systems was lighter than anticipated, there were many other examples of visible damage observed in the aftermath of Hurricane Andrew. For example, a drifting rig wiped out 5 wells in 11 m of water. The wells were found bent approximately 70° to the vertical. Bent caisson-type well protectors were another common sight. In some cases, standard caissons of 76 cm in diameter and 4.5 cm wall thickness at the mudline were found to be bent 5, 12, even 70° from the vertical. Also observed was an overturned pipeline gathering facility in Ship Shoal. Other damage noted includes cracks in conductor bay bracing at 68 foot elevation at another site (MMS 1992).

Further illustrations of much of the damage commonly observed are as follows : in Figs. 17 & 18, damage to deck appurtenances and a ladder is shown. A partially lifted deck is shown in Fig. 19. It appears that the wind induced moment caused by the lifting surface of the helipad may have caused a significant overturning moment. In Fig. 20, a heavily damaged platform with missing deck module is shown. A combination of wave and wind loading appears to have caused this platform to fail. In Figs. 21~23, damage to both the corrugated walls and roof of an enclosure around deck equipment is chronicled. The damage scenario, particularly in the case of the ridge row, is reminiscent of a very commonly observed situation for land-based structures, where loss of connections either due to corrosion or rupture of corrugated sheeting at the fastener causes an opening. This is then followed by progressive failure that is often caused by an increase in internal pressure coupled with a higher negative pressure over the building envelope (Kareem 1986). A more detailed analysis of specific failures in the Gulf during Andrew that takes into account the age and number of storms experienced in lifetime, design loads and analysis procedures, and information on wind and wave fields would provide answers to many unanswered questions. Following the hurricane, the MMS issued a call for mandatory inspections of all platforms located within 80 km northeast of the path of the eye and within 56 km of the southwest side. This required about 700 platforms to be inspected.

While there are many estimates of the damage incurred by Andrew, MMS estimated that the repair costs alone would reach \$200 million. This does not include, however, cost to redrill wells, lost reserves due to uneconomic repairs, deferred production or damage to shore bases and facilities. The hurricane initially halted 20% of the gas production and 33% of the oil production (MMS 1992). Some estimate the total monetary losses due to damage of offshore structures at over \$1 billion.

#### **4. Conclusions**

The discussion presented here provides an essential input to the investigation of the wind induced local and global effects on the structural integrity and operation of offshore installations, and future requalification of existing aging petroleum exploration infrastructure. Some of the key issues that warrant future investigations include the following : 1) improved quantification of wind field characteristics; 2) better modeling of the interference and shielding effects, overturning caused by lifting surfaces, and examination of the influence of local wave profile on the upflow angle, local intensification of flow and changes in the angle of flow over the deck; 3) mapping of

wind condition around the deck for human safety and operational considerations; 4) improved gust loading factors for offshore applications; 5) investigations of the influence of vortex shedding-induced vibration in flare booms and cranes; 6) examination of the effect of the combined action of wind, wave and currents; 7) analysis of parameter uncertainties on the load effects; 8) and synthesis of this information for developing enhanced engineering guidelines to assess the performance and safety of existing and future structures and to develop mitigative measures.

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