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DEVELOPMENT & VERIFICATION OF A SIMPLIFIED METHOD TO EVALUATE STORM LOADINGS ON AND CAPACITIES OF STEEL, TEMPLATE-TYPE PLATFORMS

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ABSTRACT

During the past three decades, an immense amount of effort has been devoted to development of sophisticated computer programs to enable the assessment of storm wind, wave, and current loadings and the ultimate limit state capacity characteristics of conventional, pile-supported, template-type offshore platforms. These programs require high degrees of expertise to operate properly, are expensive to purchase and maintain, and require large amounts of manpower and time to complete the analyses. Due to the sophistication of these programs and the expertise required to operate them, experience has shown that it is easy to make mistakes that are difficult to detect and that can have significant influences on the results.

This paper summarizes the development of simplified procedures to evaluate storm loadings imposed on and induced in template-type platforms and to evaluate the ultimate limit state lateral loading capacities of such platforms. Reasonable simplifications and high degrees of user friendliness have been employed in development of the software to reduce the engineering effort, expertise, and costs associated with the analyses.

Verification of these procedures has been accomplished by comparing the results from the simplified analyses with the results from three-dimensional, linear and nonlinear analyses of a variety of template-type platforms. Good agreement between results from the two types of analyses has been developed for the evaluations of both loadings and capacities.

INTRODUCTION

Requalification of existing offshore platforms involves developing an understanding of the Ultimate Limit State (ULS) lateral load capacity of the structure. Nonlinear analyses of offshore platforms are difficult and costly to perform. Given a large number of structures to be requalified, it is desirable to have a simplified method to estimate the ultimate limit state capacity of the platform.

A simplified procedure has been developed to estimate the ULS lateral load capacity of the three primary components that comprise conventional steel template-type offshore platforms: the deck legs, the jacket, and the pile foundation. In addition, a simplified procedure has also been developed to estimate the wind, wave, and current lateral loadings.

With these results, the Reserve Strength Ratio (RSR) can be determined as:

$$RSR = \frac{Ru}{S_R} \tag{1}$$

Ru is the ultimate lateral load capacity. S_R is the reference storm total maximum lateral loading. The reference lateral loading is that specified in current platform design guidelines such as API RP 2A (API, 1993).

The remainder of this paper will detail development, verification, and applications of the simplified procedures to estimate the storm loadings, the ULS capacity of template-type offshore platforms, and "fragility" curves that characterize the likelihoods of platform failure for a given storm intensity.

APPROACH

Fig. 1 summarizes the analysis process. In these analyses, an attempt is made to use "unbiased" estimates of the parameters that determine both loadings and capacities. A Personal Computer program identified as ULSLEA (Ultimate Limit State Limit Equilibrium Analyses) has been developed to perform the analyses.

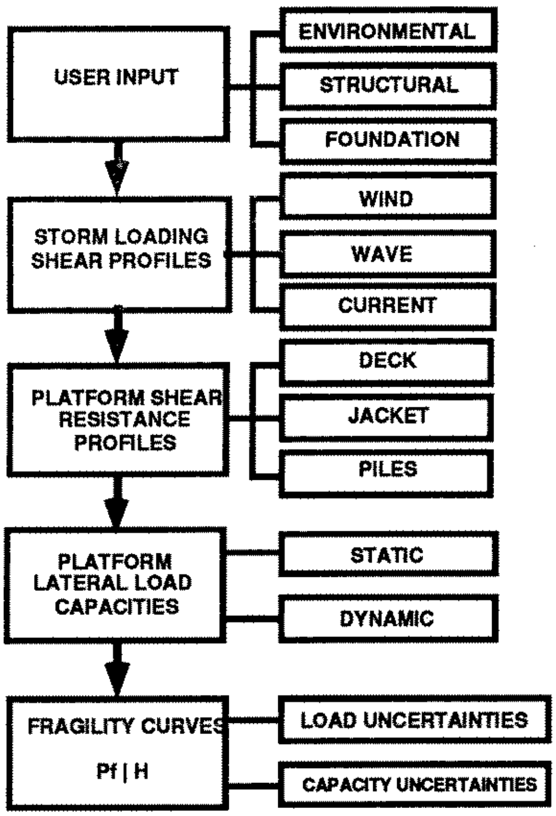

FIGURE 1 - PLATFORM LOADING, CAPACITY AND PROBABILITY OF FAILURE ANALYSES

Fig. 2 illustrates the approach used to compute the static capacity of the platform system. A series of storm loading profiles of horizontal shear are developed. The platform elements and appurtenances are modeled as a series of equivalent vertical cylinders located at the wave crest position. Block volumes are used to model the deck elements. Simplified methods are used to estimate the wave kinematics. Standard API methods 919930 are used to estimate the wind, wave, and current loadings based on user specified wind speed, wave height and period, and current velocities. Loadings from inundation of the lower decks by the wave crest also may be determined.

A profile of horizontal shear capacity of the platform is developed. The horizontal shear capacities of the deck legs, each of the bays in the vertical truss system that comprise the jacket, and the piles (axial and lateral) are determined based on the ultimate limit state capacities of the elements that comprise these components. Brace capacities depend on the direction of loading (tension, compression), and the capacities of the joints at the ends of the braces. Provisions are made for local loadings from waves and currents, deck loading P-Δ effects, moment induced shears at the top and bottom of the jacket, and the shear resistance developed by the battered jacket legs and piles.

Comparison of the storm shear profile with the platform shear capacity profile identifies the "weak link" in the platform system. The base shear or total lateral loading at which the capacity of this weak link is exceeded defines the static lateral capacity of the platform (Rus).

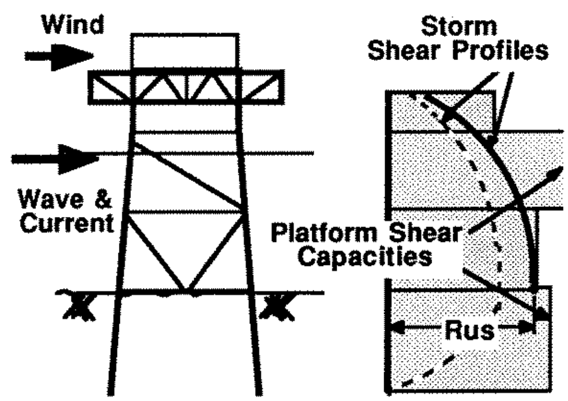

FIGURE 2 - ULTIMATE LIMIT STATE LIMIT EQUILIBRIUM ANALYSIS TO DEFINE PLATFORM STATIC CAPACITY

The static lateral loading capacity is corrected with a "loading effects" modifier (Fv) [Bea, 1991; Bea, Young, 1993] to recognize the interactive effects of transient wave loadings and nonlinear hysteretic platform response; thus

$$Ru = Rus\,(F_v) \tag{2}$$

Once the best estimate capacity has been determined, the platform probability of failure (Pf) is determined conditional on the occurrence of a given storm intensity. Storm intensity is based on the expected maximum wave height (H) with the other storm parameters (wind speed, current velocities) conditional on the time and direction of occurrence of H. The storm intensity is expressed with the total maximum lateral force developed on the platform by the storm ($S \mid H$).

Uncertainties in the loadings (σ_S) and capacities (σ_{Ru}) and the correlation (ρ) between Ru and S are specified and the conditional probabilities of failure ($Pf|H$) determined (Bea, 1990). To determine the probability of failure for all storm intensities (Pf), the conditional probabilities of failure are multiplied by the probabilities of experiencing a given storm intensity ($P|H$) and summed over all storm intensities (Bea, et al., 1994).

EXAMPLE

The analyses outlined in Fig. 1 will be illustrated with application to a four-leg platform (Fig. 3) that was installed in a water depth of 73 m in 1984. The platform supports a 34 MN deck weight and has two boat landings. The cellar deck is located at +10.6 m;. The jacket legs are 1.2 m in diameter and the 1.0 m diameter piles are grouted inside the legs. The jacket braces range from 41 cm to 61 cm in diameter. The sea floor is covered with a l m thick layer of soft clay. Below this layer, the foundation soils can be classified as stiff, over-consolidated cohesive sediments that have an average shear strength of 100 kPa to a depth of 10 m.

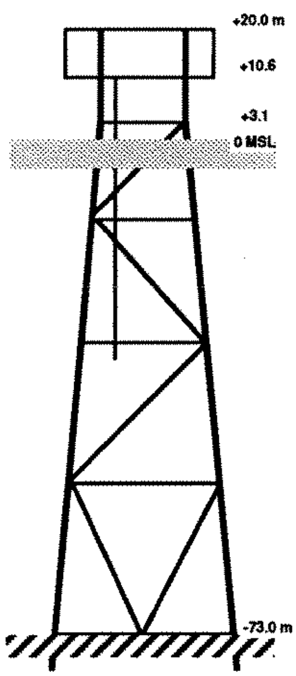

FIGURE 3 - EXAMPLE PLATFORM

The platform was designed according to 1983 API RP 2A guidelines with a 100-year design wave height of H_D = 12 m. The design criteria included storm associated currents (1.2 m / s at surface). The hydrodynamic forces were computed with the Morison formulation based on a drag coefficient of Cd = 0.75 and an inertia coefficient of Cm = 2.0. Provisions were made for marine growth accumulations on all underwater elements (10 cm). The design storm maximum total lateral force was S_D = 4.5 MN.

To perform the ULS capacity analyses, the storm conditions were specified as combinations of wind speeds and currents that were conditional on the time and direction of occurrence of a given expected maximum wave height (H). H was ranged from H = 10 to 25 m. The variation of the maximum lateral force components with H are summarized in Fig. 4. The wave loadings acting on the lower decks of the platform exceed the wind loadings for H ≥ 15 m and become equal to the wave loadings acting on the rest of the structure at H = 22 m.

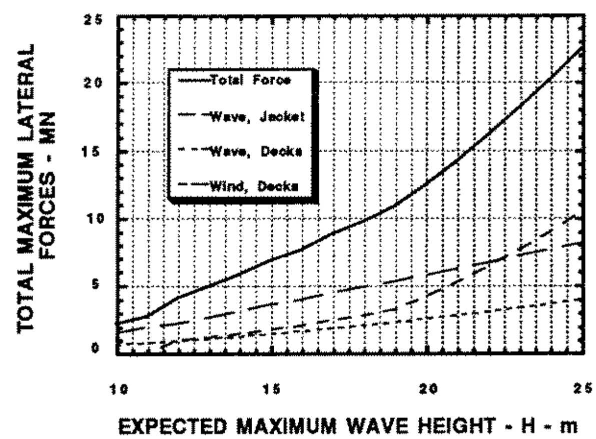

FIGURE 4 - STORM LATERAL FORCE COMPONENTS

These results were verified with results from detailed three dimensional wind - wave - current loading analyses for comparable wind, wave height, and current conditions. In general, the simplified procedure under-predicted the total maximum lateral loadings by about 20 %. The principal differences were traced to the directional spreading, shielding, and current blockage kinematics factors that were introduced into the simplified analyses. These factors had not been included in the original design loading analyses. Once these same kinematics correction factors were introduced into the detailed analyses, the simplified procedure tended to slightly overestimate the forces. This difference was traced to the lack of horizontal spatial distribution in the platform elements in the simplified analyses. All of the platform elements are concentrated at a single vertical position in the wave crest in the simplified analyses.

The vertical profile of static shear capacity of the example platform from the simplified ULS analyses is summarized in Fig. 5. The vertical profiles of shears for three storm conditions acting on the platform are also shown. The lateral shear capacity in the jacket was generally determined by the tensile - compressive brace capacity in each bay. Due to the grouted joints and heavy-wall joint cans, the joints did not control the diagonal brace lateral capacities.

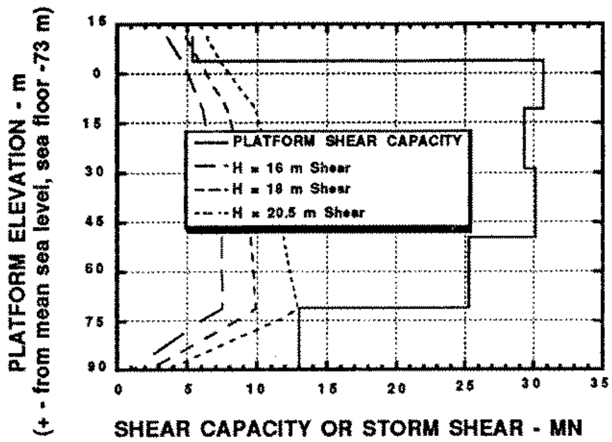

FIGURE 5 - PLATFORM STORM LOADING SHEAR PROFILE COMPARED WITH STRUCTURE SHEAR CAPACITIES

Based on the simplified loading and capacity analyses, the static shear capacity of the platform is determined by the deck legs. The best estimate deck leg shear capacity occurred when H ≈ 17 m. For this condition, the total lateral static loading and capacity is Rus = 9 MN.

However, if the deck loadings on the example platform are somewhat lower than estimated by these analyses, the static lateral capacity of the platform would be controlled by the foundation piles. In this case, the lateral shear capacity of the platform would occur when H ≈ 20.5 m. For this condition, the total lateral static loading and capacity is Rus = 13 MN.

There are large differences in the reserve strength of the components that comprise this platform. The jacket has a much larger reserve strength than either the deck legs or the foundation piles. The insights provided by these simplified loading and capacity analyses could be used to develop a more balanced design in which there would be comparable levels of reserve strength in the deck legs, jacket, and foundation piles. The simplified analyses also provide an expedient way to examine the potential effects of damage and defects on the capacity of the platform. High defect - damage probability and high lateral capacity consequence members can be defined and alternative load paths provided to minimize excessive loss of capacity.

Detailed results from nonlinear static push over analyses were developed for this platform. To perform these analyses, a nodal lateral loading pattern was developed based on a 100-year return period wave height condition (H = 12 m). The lateral loading pattern was proportionally increased to push the structure to ULS. The results are summarized in Fig. 6. The results indicate that the platform has a total lateral capacity of Rus = 12.5 MN.

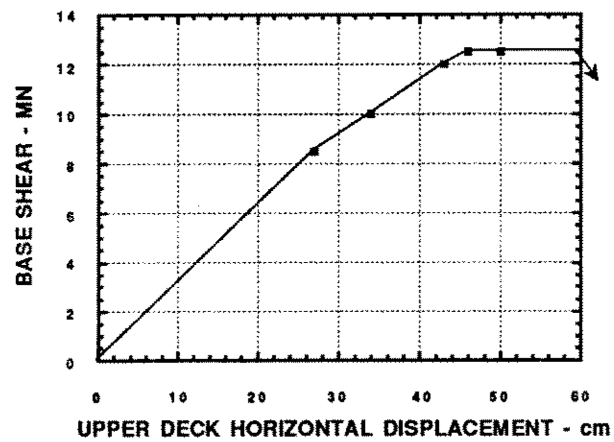

FIGURE 6 - NONLINEAR STATIC PUSH OVER ANALYSIS RESULTS

The data points shown in Fig. 6 identify the nonlinear events that developed in the platform. All of these nonlinear events were confined to the piles.

These results gave concern for the verification of the simplified capacity analysis summarized in Fig. 4. The difference between the detailed nonlinear lateral capacity (Rus = 12.5 MN) and that estimated by the simplified method (Rus = 10 MN) was traced to the lateral loading pattern that had been used to perform the nonlinear analyses.

In the case of the detailed nonlinear analysis, the loading pattern had not been changed as the wave loadings increased. In the case of the simplified analysis, the wave loading pattern was changed as a function of the wave heights. This resulted in more force at the top of the jacket due to wave crest loadings on the lower cellar deck. The failure mode was shifted from the piles to the deck legs. The simplified method identified an error in the nonlinear analysis. Once the loading pattern was adjusted as a function of the wave heights in the nonlinear analysis, the static push-over results indicated a lateral capacity of Rus = 9 MN.

The difference between the nonlinear static push-over Rus = 9 MN and the simplified ULS capacity of Rus = 10 MN was traced to neglect of the vertical loading - lateral displacement (P- Δ) moments in the deck legs in the simplified analyses. An approximate analysis of the relative lateral displacement between the bottom of the deck and the top of the jacket based on four fixed-fixed end columns free to displace at their top ends and loaded with the estimated ULS wind and wave deck loads produced a moment that was used to reduce the ULS capacity of the deck legs. This correction brought the results into good agreement.

Ambient vibration measurements performed onboard this platform indicated a natural period of Tn = 1.5 sec. The ULS wave H ≈ 17 m had a period of Tw = 12 sec. Thus, Tw / Tn = 8. The platform was capable of developing a system ductility of $\mu \approx 3$. The results developed by Bea and Young (1993) indicate a wave transient loading - nonlinear response correction factor of Fv = 1.2 . Thus, the best estimate capacity was Ru = 10.8 MN.

Fragility curves were developed that expressed the probability of platform failure conditional on a specified wave height (P | H). The uncertainties in the platform capacity and the storm loadings were evaluated as outlined in Bea (1990). Both inherent or "natural" randomness (Type I) and modeling - parameter (Type II) uncertainties were included in the analyses (Bea, 1993). The results of the analyses are summarized in Fig. 7. Inclusion of Type II uncertainties has important effects on the results.

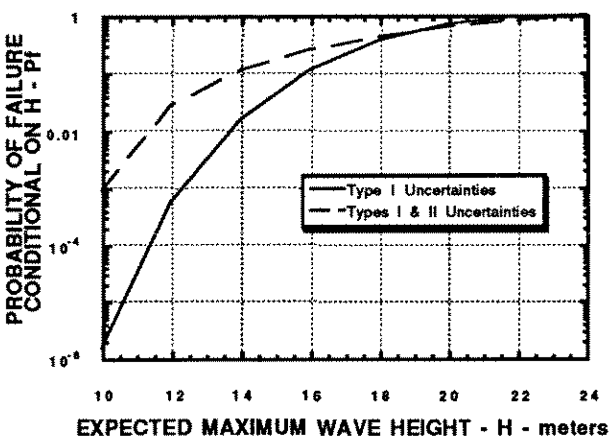

FIGURE 7 - FRAGILITY CURVE FOR EXAMPLE PLATFORM

GULF OF MEXICO PLATFORM VERIFICATIONS

Analytical and performance experience with six Gulf of Mexico (GOM) platforms has been used to verify the simplified analyses. The characteristics of these platforms are summarized in Table 1. The verification cases include four self-contained drilling and production platforms and two tender assisted drilling platforms designed and installed during the period 1959 through 1970. All were built from A-36 steel. All were founded on good soils. Design criteria ranged from 25-year to 100-year return period storm conditions. The design hydrodynamic drag and inertia coefficients were in the range of Cd = 0.5 to 0.7 and Cm = 1.5 to 2.0. Design criteria for only one of the platforms (C) included storm associated currents. Joint designs included gusseted, heavy wall joint cans, and grouted leg - pile annuli.

Several of these platforms experienced severe loadings from tropical cyclones. Two of the platforms (A, B) less than one year old were located close to the path of hurricane Hilda (1964). One of the platforms failed (A) and the other experienced significant damage (B). In a period of 15 years, platform F experienced four storms that generated wave heights that were approximately equal to or greater than its design wave height. One of the platforms (C) was in the immediate path of hurricane Camille (1969) and experienced green water in the lower decks without significant damage. There were seven other almost identical platforms in the same vicinity that also survived without significant structural damage.

Table 2 summarizes the results of the verification analyses. The results are keyed to the figure numbers in this paper. The remainder of this section will discuss the analyses.

Platform A

This structure (fig. 8) was designed for a 25-year return period wave height of H_D = 14 m without any air gap. Due to an error in determining the water depth, the platform was placed in a water depth 0.6 m greater than originally intended.

The simplified ULS capacity analysis Rus = 7.1 MN (broadside). The critical mode of failure was in the deck legs at the top of the jacket. The top row of diagonal braces were also close to failure for this condition. The API reference lateral load of S_R = 9.4 MN includes a wave crest loading on the lower cellar deck of the platform of 2.2 MN.

The results of the nonlinear push-over analyses of this platform are summarized in Fig. 9. The ULS failure mode involved failure of the deck legs and top bay of diagonal braces. This failure mode was obtained only when the dynamic stiffness and capacity characteristics of the foundation were recognized and biases removed from the evaluations of the pile lateral and axial capacities (Bea, 1992a). The use of conventional static capacity analysis methods under-predicted both foundation stiffness and capacity and indicated that the failure mode was initiated and constrained to the foundation piles.

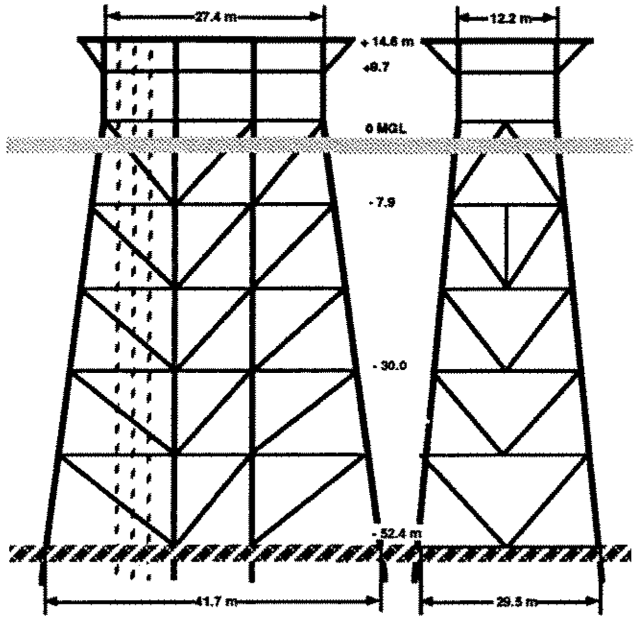

FIGURE 8 - ELEVATIONS OF PLATFORM A

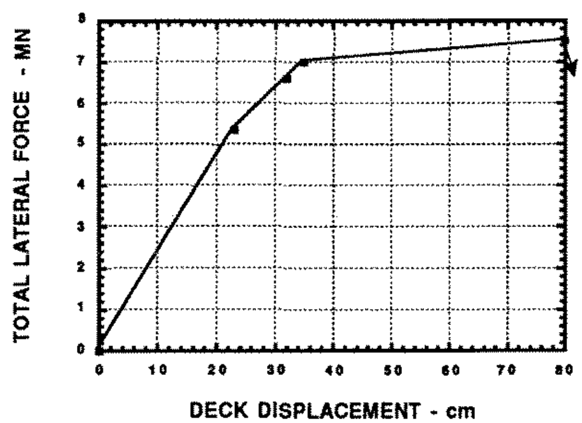

FIGURE 9 - PLATFORM A STATIC PUSH OVER ANALYSIS RESULTS

This platform failed one year after it was installed (Fig. 10) [Hilda Meeting Transcript, 1964]. The failure was apparently triggered in the deck legs and top bay of diagonal braces. Based on the hindcast hurricane conditions that were present at the platform [Bea, 1974], the maximum total lateral loadings were estimated to be S_M = 9.0 MN (maximum wave height Hm = 18 m). The maximum wave resulted in significant inundation of the lower deck.

Recognizing that the platform had a natural period Tn ≈ 1.8 sec and that the period of the maximum waves were in the range of Tw = 10 to 11 sec, Tw / Tn = 5.6 to 6.1. Given that the platform could develop a ductility of $\mu \approx 3$ (Fig. 9) then Fv ≈ 1.2 The best estimate dynamic capacity would be Ru ≈ 8.5 MN. The computed peak storm loading exceeded the computed platform capacity.

The platform should have failed. It did, and in the way predicted by the simplified loading and capacity analyses.

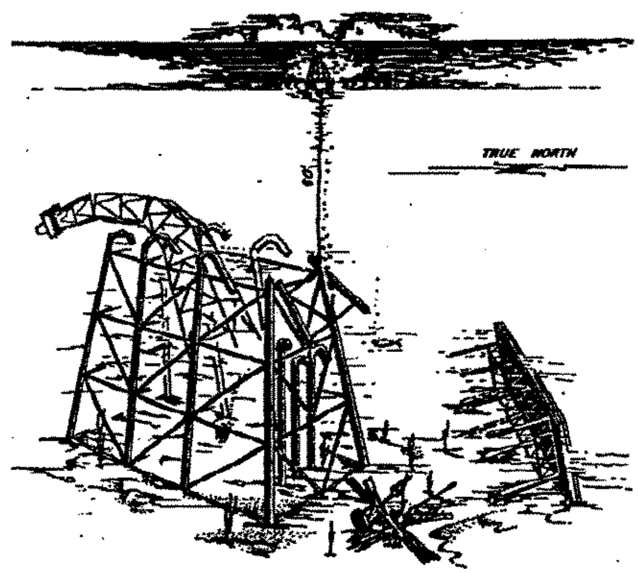

FIGURE 10 - PLATFORM A AFTER HURRICANE HILDA

Platform B

This is a nearby similar platform (Fig. 11) installed in a water depth of 66 m in 1963. The platform was designed for a 100-year return period storm with a maximum wave height of H_D = 17 m with an air gap of 1.5 m. The design hydrodynamic forces were determined based on Stokes Fifth Order Theory kinematics, no current, and a drag coefficient Cd = 0.5. The total lateral design loading was S_D = 8 MN [Marshall, Bea, 1976].

Application of the simplified analyses indicated Rus = 20 MN for the broadside loading condition. The critical mode of failure involved compressive buckling in the top three levels of vertical and horizontal diagonals. The end-on loading condition indicated a capacity that was 25 % larger than for the broadside condition (Rus = 25 MN).

The results of the detailed nonlinear analyses of this platform indicated Rus = 19.1 MN (Fig. 12). As for the platform A analyses, these lateral capacity analyses considered the pile foundation dynamic loading characteristics and removal of biases from the axial and lateral capacity evaluations. The failure mode involved the top two bays of vertical diagonal braces.

TABLE 1 - VERIFICATION PLATFORM CHARACTERISTICS & STORM EXPERIENCE

Plat ID	Year	Water Depth m	No. Legs / Piles	Lower Deck El +m MGL	Joints	Soils	Storm Experience
A	1964	52.4	8 / 8	9.2	gusset	10 m soft clay over stiff clay	Hilda (1964), Failed
B	1963	66.2	8 / 8	12.8	hvy wall, gusset	as A	Hilda, Damaged
C	1968	98.2	8 / 16	13.1	heavy wall	sands and stiff clay	Camille (1969), No Damage
D	1970	82.6	8 / 8	14.0	grout	as C	Frederic (1979), No Damage
E	1959	15.9	8 / 8	11.9	grout	1 m soft clay over stiff clay	Carla (1961), Beulah (1967), Alicia (1985), Damaged
F	1963	42.7	5 / 5	10.4	grout	3 m soft clay over stiff clay	Carla (1961), Hilda (1964), Celia (1970), Carmen (1974), Betsy (1979), Allen (1980), Damaged

TABLE 2 - SUMMARY OF LOADING - CAPACITY VERIFICATIONS

Plat. I. D.	S_D MN	S_R MN	S_M MN	Rus MN	Fn	Ru MN	RSR	Rus nl	B_{Rus}
A (fig 8)	5.6	9.4	9.0 (fig 10)	7.1	1.2	8.5	0.90	7.4 (fig 9)	1.04
B (fig 11)	8.0	12.0	8.6	20.0	1.2	24.0	2.0	21.4 19.0 (fig 12)	1.07 0.96
C (fig 13)	15.0	21.0	25.0	26.0	1.25	32.5	1.55	27..0 (fig 14)	1.03
D (fig 15)	11.3	10.3	9.0	14.0 (fig 16)	1.2	16.8	1.63	12.5 (fig 17)	0.89
E (fig 18)	3.1	4.0	4.5	4.5 (fig 19)	1.0	5.0	1.0	3.6 (fig 20)	0.80
F (fig 21)	3.6	9.0	8.1	5.0 (dam) 6.0 (rep) (fig 22)	1.0 1.0	3.2 5.9	0.36 (dam) 0.66	4.5 5.4 (rep) (fig 23)	0.9 0.9

S_D - design lateral loading, S_R - API Reference lateral loading, S_M - maximum loading experienced, Rus - Simplified analysis lateral capacity, Fn - transient loading - nonlinear response correction factor, Ru - best estimate lateral capacity, RSR - Reserve Strength Ratio, Rus nl - lateral capacity based on nonlinear analyses, B_{Rus} - Bias (Rus nl / Rus) in simplified analysis

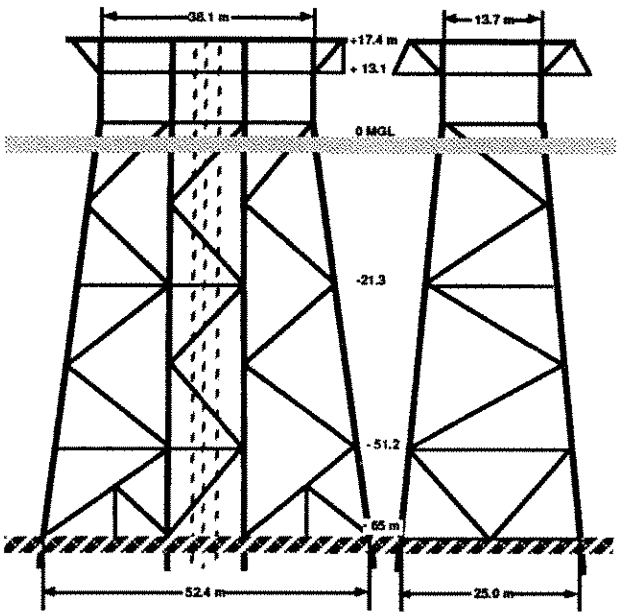

FIGURE 11 - ELEVATIONS OF PLATFORM B

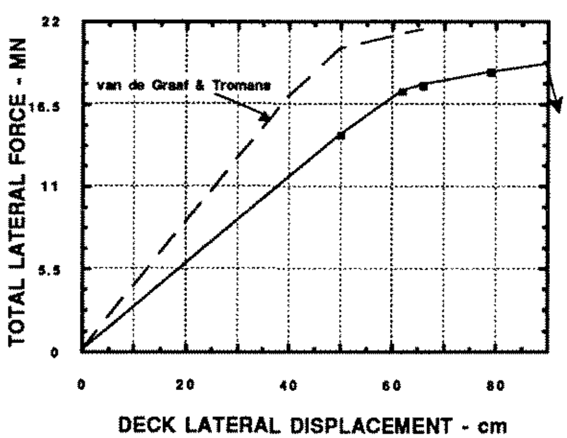

FIGURE 12 - STATIC PUSH OVER RESULTS FOR PLATFORM B

These lateral capacity estimates were confirmed with results from nonlinear push-over analyses published by van de Graff and Tromans (1991) (Fig. 12). Their results indicated Rus ≈ 21.4 MN for the analyses that suppressed lateral pile failure. The failure mode for this condition was concentrated in the vertical diagonal braces. The analyses that utilized conventional pile capacity characterizations indicated Rus ≈ 16.5 MN. The failure mode for this condition was concentrated in the piles.

Note the differences produced by the two nonlinear analyses (Fig. 12); there is about a 12 % difference in the predicted Rus. The major part of this difference is due to the various assumptions that are made in the two analyses regarding the nonlinear characteristics of the braces and joints.

This platform survived the same storm that resulted in the failure of Platform A. The storm wave crests did not reach the lower decks. The maximum lateral loadings were estimated to be S_M = 8.6 MN (Bea, 1974; Marshall, Bea, 1976). The vertical diagonal braces in the platform were extensively damaged; two of the jacket legs were parted (Hilda Meeting Transcript, 1964). The platform should have survived and it did. The platform damage was repaired and this structure is in service today.

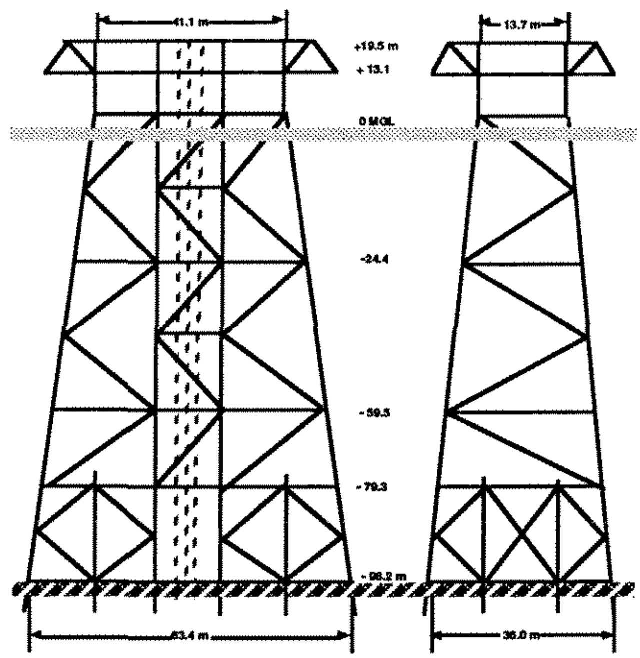

FIGURE 13 - ELEVATIONS OF PLATFORM C

Platform C

This platform (Fig. 13) is a more recent 8-leg, 12-pile structure that was designed for a 100-year storm wave height of H_D = 18 m with an air gap of 1.5 m, the storm associated currents (1.2 m / s at surface), and Cd = 0.5. The platform was installed in 1968 and was designed according to the draft guidelines of the first API RP 2A.

Application of the simplified ULS capacity analysis gave Rus = 26 MN for broadside loading conditions. The primary mode of failure involved compressive buckling in the third and fourth levels of vertical diagonal bracing. The broadside capacity was approximately 70 % of the end-on loading capacity (Rus = 37 MN).

These lateral capacity estimates are in reasonable agreement with those originally estimated for this platform (Marshall, Bea, 1976]) This earlier work indicated Rus = 30.2 MN (ductile redundant with wave in deck) to Rus = 36.7 MN (ductile redundant without wave in deck). The Reserve Strength Ratios were estimated to be in the range of RSRs = 2.1 to 2.5. The best estimate capacity was taken as Rus = 30.2 MN. These results are remarkable when it is remembered that they were developed using linear structure analyses.

Analyses published by Tromans and van de Graff (1992])confirm these static capacity results (Fig. 14). Their work indicates Rus = 38.6 MN (for end-on loading condition) to Rus = 27.0 MN (for broadside loading condition).

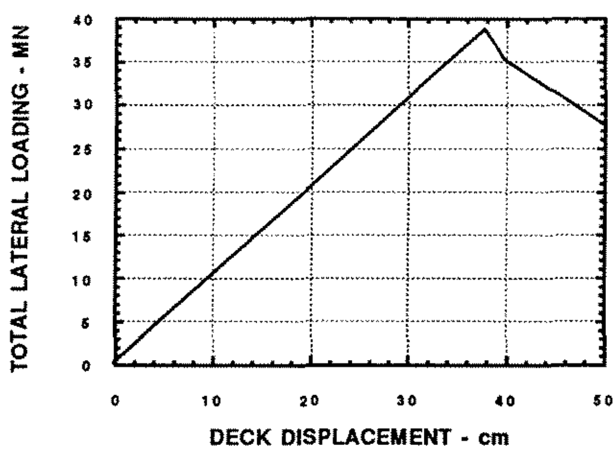

FIGURE 14 - STATIC PUSH OVER RESULTS FOR PLATFORM C

Ambient vibration measurements performed onboard this platform indicated that the natural period of this platform is approximately Tn = 1.5 sec (Ruhl, 1976]) The maximum wave heights in hurricane Camille had periods in the range of Tw = 11 to 12 sec. The ratio Tw / Tn = 4.4 to 4.8. Recognition of transient loading - nonlinear capacity performance effects indicates a loading effect factor of Fv = 1.25 (Bea, Young, 1993). This evaluation agrees well with results recently published by Stewart (1992])for a comparable platform, transient wave loading conditions, and nonlinear - dynamic response.

This platform, and seven other similar structures survived the intense portion of hurricane Camille. One of the platforms recorded a 22 m wave height before the wave staff failed. Several of the platforms indicated substantial wave crest damage in the lower decks (Bea, 1974). The maximum wave heights in the storm were estimated to be approximately H_M = 24 m. The maximum total lateral loading estimated on this group of platforms ranged from S_M = 20 to 25 MN (Marshall, Bea, 1976; Stewart, et al., 1988]) All of the platforms survived without substantial damage. The simplified loading and ULS capacity analyses indicate that they should have.

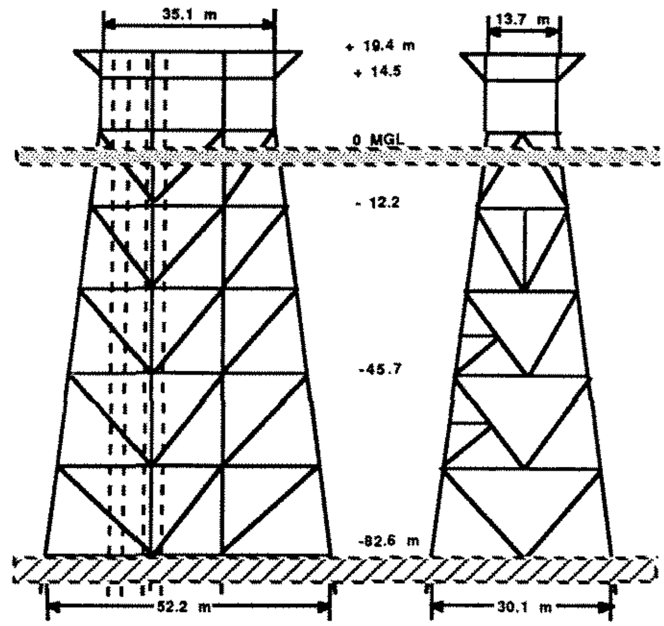

FIGURE 15 - ELEVATIONS OF PLATFORM D

Platform D

This structure (Fig. 15) is the most recent of the 8-leg GOM platforms studied. Like platform C, the platform was designed according to the first edition API RP 2A guidelines for 100-year storm conditions that included a design wave height H_D = 17.7 m, currents (1.1 m / s at surface), an allowance for marine growth, and Cd = 0.6 to 0.7 (function of member diameter).

The results of the simplified loading and capacity analyses are summarized in Fig. 16 (Bea, 1992b, 1992c). The 100-year storm lateral loading shears as a function of elevation are compared with the shear capacities of each of the bays in the jacket and the deck legs. In this case, because of the vertical diagonal brace framing patterns, the end-on loading capacity is less than the broadside loading capacity. Both capacities are governed by the diagonal brace and leg shear capacities in the fourth level of bracing below the jacket top.

Results from nonlinear push-over analyses of this platform are summarized in Fig. 17. The push-over analyses indicate that the vertical diagonals and several of the horizontal members in the top four bays of the jacket are involved in the failure mode. In this case, the simplified method over-estimated the capacities.

The tendency to over-estimate the capacities was traced to neglect of the local wave pressure induced moments in the upper levels of bracing. Corrections were introduced to the brace compressive capacities to recognize the local wave pressures (reduced capacities by 10 to 20 %). Once this factor was introduced into the calculation of the vertical diagonal brace compressive capacities, the results agreed very closely.

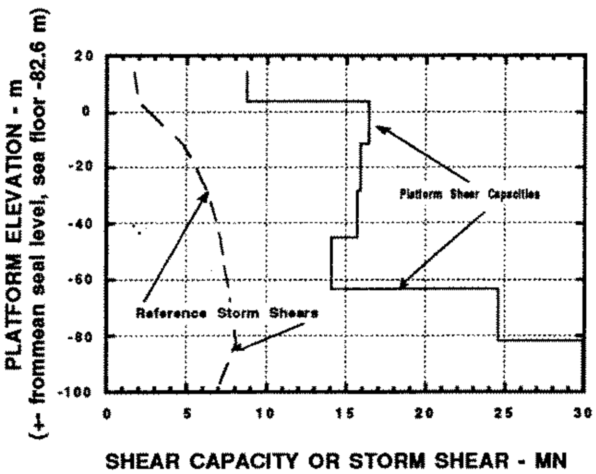

FIGURE 16 - PLATFORM D 100-YEAR STORM CONDITION SHEAR PROFILE AND STRUCTURE SHEAR CAPACITY

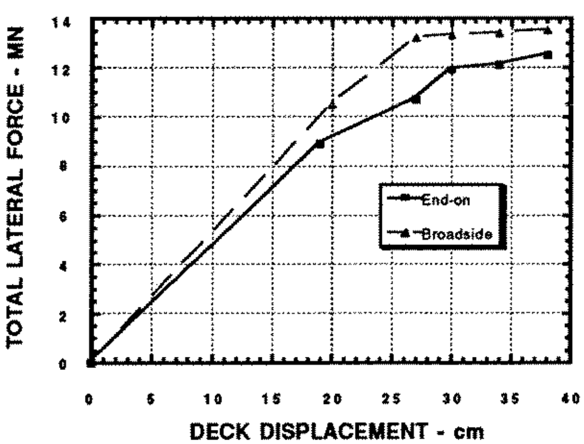

FIGURE 17 - PLATFORM D STATIC PUSH OVER ANALYSIS RESULTS

This platform was located on the east side of the Mississippi River delta. In 1988, this structure experienced maximum wave heights during hurricane Frederic of H_M = 15 m. The maximum total lateral storm force was estimated to be $S \approx 9$ MN. The platform survived without damage. The simplified analyses indicate that it should have.

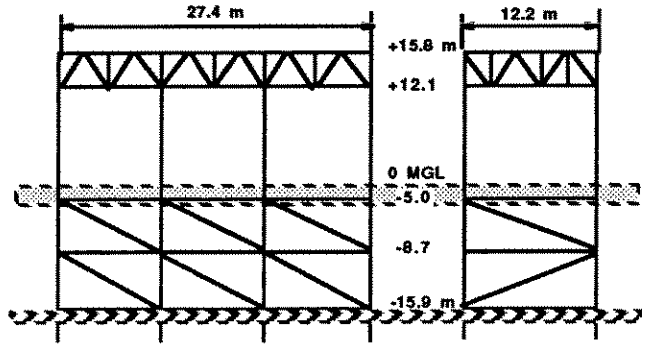

FIGURE 18 - ELEVATIONS OF PLATFORM E

Platform E

This platform (Fig. 18) was installed in 1959 in 15.9 m of water. The 8-leg tender drilling assisted platform had 83.8 cm diameter, 1.3 cm wall thickness legs inside of which were grouted 76 cm diameter piles. It was single diagonal braced with 32.4 cm diameter members that were battered in the same direction. The jacket was placed in a water depth that was 4.6 m deeper than intended. For that reason, the top of the jacket was at elevation -1.5 m. The platform was designed for 25-year wave height criteria with H_D = 11.6 m. The present API reference level wave height is H_R = 13.4 m.

The simplified loading analysis (Fig 19) indicated a total lateral loading associated with the 100-year API wave height and forces condition to be S_R = 4.5 MN. It was this condition that brought the platform to ULS for broadside loading. The failure mode was concentrated in the deck legs for both the end-on and broadside loading conditions. There is a marked difference in the horizontal loading capacities of the structural components that comprise the jacket. The deck legs are the weak link in this platform structure system. Diagonal bracing of the deck legs could be very effective in rehabilitating this structure.

Detailed nonlinear analysis results have been developed for broadside and end-on loading conditions (Fig. 20) (Bea, 1992c). These results indicate Rus = 3.6 MN for the broadside loading and Rus = 5.0 MN for the end-on loading. The nonlinear analysis did not indicate that the deck legs had comparable shear capacities for end-on and broadside loadings.

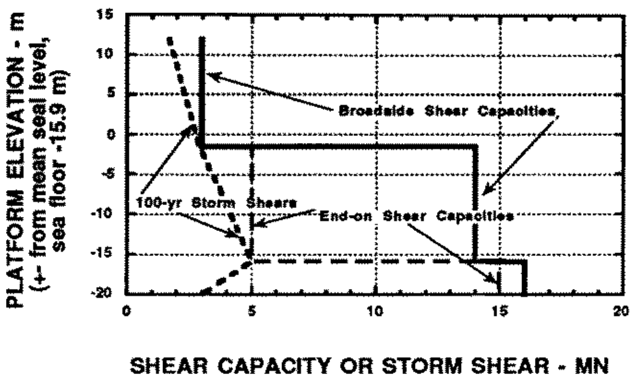

FIGURE 19 - PLATFORM E 100-YEAR STORM SHEARS AND BROADSIDE AND END-ON SHEAR CAPACITIES

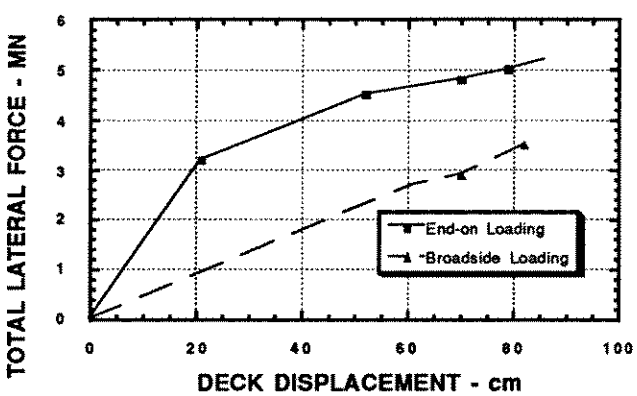

FIGURE 20 - STATIC PUSH OVER RESULTS FOR PLATFORM E

Given the low natural period for this platform (Tn < 1 sec), the transient loading - nonlinear response corrections to Rus are insignificant (Fv ≈ 1.0). Thus, Ru ≈ Rus.

Hurricane Carla subjected this platform to waves that developed maximum lateral forces of approximately S_M = 4.5 MN. It is likely that these waves approached this platform end-on. The storm forces were less than the platform capacity. The platform was damaged but did not fail during this storm. The analytical results are in conformance with this observation.

Platform F

This platform (Fig. 21) is a 5-leg (4 corner, 1 center), tender drilling assisted platform that was located in a water depth of 45.7 m in 1963. The leg - pile annulus was ungrouted and the piles attached to the jacket with welded shimmed connections at the top of the jacket.

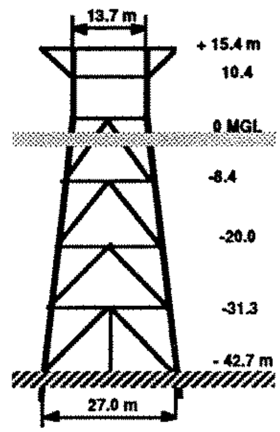

FIGURE 21 - PLATFORM F ELEVATION AND PLAN

The platform was extensively damaged during four hurricanes that produced wave heights at the location that equaled or exceeded the 25-year design wave height of H_D = 14 m (Bea, et al., 1988). The damage included missing diagonal braces and cracked joints. There was extensive damage to one leg of the platform that was developed when acid from the well workovers was repeatedly allowed to leak onto the leg. One of the broken diagonal braces was attributed to a compressor that had been accidentally dropped overboard during its installation.

The present API 100-year wave height is H_R = 20 m. Fig. 22 shows the 100-year storm loading shears and the platform shear capacities. The platform shear capacities for two conditions are shown; one for the as-is condition and one for the damage to the joints and braces repaired and the leg-pile annulus grouted. The effects of the repairs and grouting are relatively small. This is because the failure mode is concentrated in the deck legs.

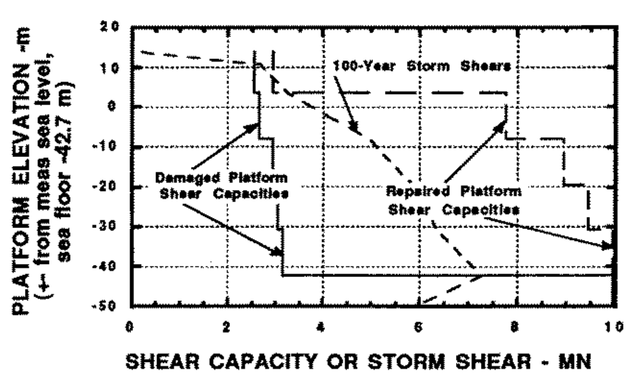

FIGURE 22 - PLATFORM F 100 YEAR STORM SHEARS AND SHEAR CAPACITIES

Nonlinear analyses have been performed on this platform (Fig. 23) (Bea, et al., 1988) for the as-is condition and the repaired / grouted condition. The simplified analyses tend to slightly over-predict Rus for this platform. The simplified analyses do a good job of predicting the capacities for both the damaged and the repaired conditions.

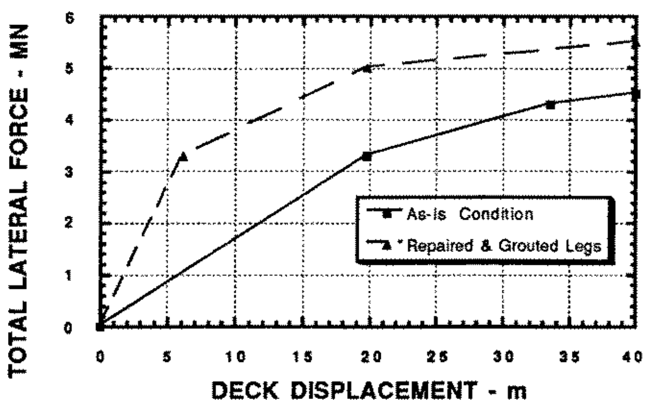

FIGURE 23 - STATIC PUSH OVER RESULTS FOR PLATFORM F

SUMMARY AND CONCLUSIONS

The results summarized in Table 2 indicate that the simplified analyses can develop evaluations of platform static lateral capacities that are good approximations of those derived from detailed nonlinear analyses. The simplified static capacity bias (B_{Rus} = nonlinear Rus / simplified Rus) for the seven verification cases discussed in this paper ranges from $B_{Rus} = 0.80$ to 1.03 with a mean value of $B_{Rus} \approx 0.95$.

Although not discussed in detail in this paper, the simplified analyses of storm wind, wave, and current loadings are in good agreement with results from detailed analyses. The simplified analyses are generally within ± 10 % of the detailed results as long as the same input is used for the structure characteristics, environmental conditions, and force computations.

Comparisons of the estimated lateral load capacities with the estimated maximum loadings that these platforms have experienced and with the observed performance characteristics of these platforms indicates that the analytical evaluations of both storm loadings and platform capacities are in good agreement with the experience.

The use of the simplified analytical procedures to estimate reference storm lateral loadings, platform capacities, and Reserve Strength Ratios are indicated to result in reasonable estimates that can be used for the reassessment of existing platforms. Results from the simplified analyses can be used to help validate results from the complex nonlinear analyses. In addition, the approaches outlined in this paper offer significant promise as preliminary design tools to help engineers better proportion lateral load capacity and damage - defect tolerance (robustness) in offshore platforms.

At the present time, a joint industry-government sponsored research project is underway to further develop and verify the simplified ultimate limit state limit equilibrium analysis procedures. Additional verifications are being performed on platforms, well protectors, and caissons that failed and survived during hurricane Andrew. Results from these verifications will be reported in the near future.

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