OTC 8418

Ultimate Limit State Capacity Analyses of Two Gulf of Mexico Platforms

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This paper was presented at the 28th Annual OTC in Houston, Texas U.S.A., 5-8 May 1997.

Abstract

The study summarized in this paper has two main goals: (1) verify the utility of static pushover analysis in accurately assessing a steel template-type offshore platform's ultimate limit state (ULS) behavior, and (2) provide results by which key assumptions for the program ULSLEA[1] can be verified. ULS evaluations were performed for two Gulf of Mexico platforms: South Pass 62A and Ship Shoal 274A. The results of these evaluations were compared to the historical performance of the platforms during hurricane Camille (SP 62A) and hurricane Hilda (SS 274A) in order to determine the accuracy of the solution approach. Biases and effects which skewed the analysis results away from "realistic" or historical performance were identified and studied. Also, the effects of horizontal framing in the jacket on first member failure and load redistribution were also examined.

Two major sources of bias which were examined were those affecting pile foundation modeling (sampling, testing and load-rate biases), and bracing members (calibration of buckling performance). The effects of material strength values (true yield strengths, strain-rate effects), proper load apportionment, and dynamic load reduction on overall platform performance were also briefly discussed.

Initial analyses for both structures were based on conventional design-basis foundation pile-soil interaction characteristics. In both cases, the use of these characteristics resulted in unrealistic performance of the platforms: the platforms were predicted to fail when they did not and to fail in a manner inconsistent with post-hurricane inspections. Realistic characterizations of the influences of soil sampling, testing, and load rates on pile foundation performance produced results which were in agreement with the observed behavior of the platforms.

Biases associated with material strengths and bracing member performance were found to influence the estimated maximum lateral load capacities of the structures on the order of 20-30 %. Consideration of the dynamic responses of the platforms resulted in effective static loads 10-20 % less than those established without accounting for the time-varying nature of the loads.

The difficulties in predicting local failures such as local buckling and cracking were also discussed. Local failures found in SS 274A following hurricane Hilda did not manifest themselves at the level of detail initially considered within the analyses; more-detailed evaluations of the stresses within members and at joints were needed in order to determine the failure modes. Consideration of biases in material properties and identification of stress concentrations must be made for assessment of local failures.

The effects of horizontal framing in the jacket of SP 62A on first member failure and post first member failure was studied. It was found that the presence of this framing had little effect on first member failure; however, horizontal framing was found to greatly influence load redistribution following the failure of the first member.

Results obtained for the two structures indicate the validity of using static pushover analysis to assess the lateral load capacity of steel template-type platforms subjected to hurricane wind and wave loads. With proper accounting for biases, dynamics and local stress effects, the analytical estimates were found to compare favorably with the historical performance of both structures. Hence, it is essential that the identified biases, dynamics, and local stress effects be considered within the analysis process if realistic ULS performance estimates are to be obtained.

Introduction

For the past six years, the *Marine Technology and Management Group* at the University of California at Berkeley has performed a series of research projects, sponsored by regulatory agencies and platform owners, intended to develop and refine analytical screening methodologies that can be used to assess the suitability of aging offshore structures for continued serv-

ice. This research has concentrated primarily on the development and use of ultimate limit state (ULS) analysis techniques for the purpose of screening steel template-type platforms in the Gulf of Mexico subjected to hurricane wind and wave forces. Parts of the research effort have been directed towards the development of "simplified" ULS analysis procedures[1], while other parts have focused upon performing detailed ULS evaluations of typical platforms using current analysis methodology and existing non-linear analysis software.[2] This latter effort has concentrated on verifying the suitability of using detailed static pushover analyses to make accurate assessments of a given platform's ULS behavior, and to identify conservatisms, load and structural effects not accounted for, as well as general difficulties in the performance of these analyses which may skew or bias the results away from actual platform behavior.

This paper details ULS evaluations performed using static pushover analysis of two steel template-type structures subjected to hurricane wind and wave loads: South Pass (SP) 62A and Ship Shoal (SS) 274A. The ULS behavior of each structure as predicted by the analyses is compared with platform historical data. In addition to these ULS evaluations, two analytical issues, namely the difficulties of modeling pile behavior and the effects of member imperfection on the ULS, are discussed. Furthermore, the effect of horizontal framing members within jacket structures on ULS behavior is examined.

This study had two primary goals. The first is to continue the effort begun by Loch and Bea[2] in verifying the utility of static pushover analyses in accurately assessing the ULS performance of steel template-type structures subjected to hurricane wind and wave loads. This includes not only performing additional verification studies, but also examining areas of the analysis process which may be conservative or extremely sensitive to input. The second goal is to provide detailed results on typical platform ULS behavior for use in calibrating and verifying the program ULSLEA[1], which has been developed as part of the simplified ULS analysis screening effort.

Information on each platform's structure and equipment was provided by Shell Oil Company. The results of the analyses are compared to data on actual platform performance collected following extreme loading events; SS 274A survived hurricane Hilda in 1964 with moderate damage, while SP 62A survived hurricane Camille in 1969 with no significant structural damage.

Two issues addressed in this paper, those of the modeling of the foundation piles and the effects of member imperfection on ULS behavior, are continuations of studies started by Loch and Bea.[2] The reader is referred to this report for further details concerning these studies. Results from both SP 62A and SS 274A are discussed with respect to the modeling of the foundation piles, while the model for SP 62A was used to evaluate the effects of member imperfection on ULS. The evaluation of the effects of horizontal framing on ULS behavior was intended to verify a key assumption in the program ULSLEA approach involved with determination of the lateral capacities of jacket bays. The model for SP 62A was used to perform this study.

Approach

The platforms studied were modeled using DNV's PREFRAME program.[9] Only the major structural components were included within the models; the contribution of appurtenances and conductors to the platforms' stiffnesses and strengths were neglected. However, loads induced on the platforms due to these non-structural components were taken into account. Foundation pile behavior was taken into account through the use of non-linear spring-to-ground elements.

For each platform, wind, wave and current conditions were established in accordance with API RP 2A Section 2.3.2 and Section 17.[10,11] Jacket and conductor loads were generated using DNV's program WAJAC.[12] WAJAC computes loads on members using the Morison formulation. Deck loads due to wind and wave forces were calculated in accordance with the methods outlined in API RP 2A Section 17.[10] For each platform, only the principal directions of loading (end-on and broadside) were considered; it was judged that these results should bound the true ULS of each structure in so far as failure in the jacket braces and/or deck legs was concerned.

The static pushover analyses were performed using SINTEF's USFOS program.[3-6] USFOS is a very capable advanced state-of-the-art analysis program designed to perform collapse analysis of steel offshore structures. Its solution procedure accounts for both geometric and material non-linearities. USFOS incrementally increases the load on the structure until collapse in achieved or until a global instability (such as a member buckling) occurs. In the case of global instability, the load is reduced until equilibrium is regained, at which point the load is increased again.

For each analysis, the wave height used in determining the collapse load was adjusted until a load factor of unity was achieved. Wave forces on decks and appurtenances were recalculated for each wave height using water particle kinematics appropriate to the deck and appurtenance locations. It was felt using the wave load which resulted in a load factor of unity was much more appropriate than simply increasing a lower initial wave load until collapse was achieved; the latter approach leads to concentration of forces in the lower portions of the jacket, which may result in missing the actual failure mode of the platform.

It should be noted that no dynamic effects have been explicitly considered in these analyses.[7,8] Geometric (i.e. stress concentrations) or material imperfections which might lead to cracking or brittle fracture and local member failures such as local buckling and tearing have not been explicitly considered.

Data from the analyses was collected and processed using SINTEF's POSTFOS program.[6] POSTFOS extracts data selected by the user for global, element and nodal force/displacement histories, and saves them in spreadsheet

form so that they may be easily manipulated by the user. Use was also made of SINTEF's graphical postprocessor XFOS[6] to quickly view results of the analyses and study the progress of the solution. The preprocessing, analysis and data collection were all performed on an IBM RISC 6000 computer. Some data reduction and graphics work was performed on an Intel Pentium machine using the spreadsheet Microsoft Excel 5.0.

For additional information on the program system used, and the general methodology and theoretical background of the loadings and analysis, the reader is referred to Loch and Bea.[2]

South Pass 62A

Platform Characteristics. SP 62A is one of three similar platforms located in the South Pass region of the Gulf of Mexico offshore Louisiana. It was installed in 1967. The platform is an 8-leg jacket structure sited in 340 ft of water (Fig. 1). It is classified as an unmanned drilling and production platform, and it supports eighteen 24 in.-diameter conductors. The platform was originally designed in 1966 for a 58 ft wave with 4 ft/sec current, and Morison drag and inertia force coefficients of 0.5 and 1.5, respectively.[13]

SP 62A has decks at +45 ft and +60 ft MGL (Mean Gulf Level). The base dimensions of the jacket are 202 ft by 122 ft, with the long dimension running NNW-SSE. The perimeter framing of the jacket is battered to 1:10. The jacket legs are fabricated from sections ranging from 53 in. in diameter (wall thickness, w.t. = 0.625 in.) to 54 in. in diameter (w.t. = 1.5 in.). The skirt guides are 56 in. in diameter (w.t. = 0.625 in.). Braces in the jacket range from 30 in. in diameter (w.t. = 0.625 in.) in the lower bay to 24 in. in diameter (w.t. = 0.5 in.) in the top bay. Most of the major joints in the jacket are canned, with the cans and brace ends (typically 5 ft on either end) fabricated from A441 steel as opposed to A36, which constitutes the majority of the jacket and piles.

The foundation consists of 8 main piles and 8 skirt piles. The main piles are 48 in. in diameter (w.t. = 0.625 in. to 1.125 in.); they pass through the legs and are welded off at +20 ft MGL. The deck legs are 48 in. in diameter (w.t. = 1.0 in.) and are welded to the tops of the main piles. The skirt piles are also 48 in. in diameter (w.t. = 0.625 in. to 1.25 in.), and are grouted in their guides in the bottom jacket bay. All piles were driven to 180 ft penetration. Based on on-site soil borings, the soil profile is characterized as:

0-12 ft	silty fine sand with clay seams
12-60 ft	silty sand (dense)
60-90 ft	fine to medium sand (dense)
90+ ft	fine to silty-fine sand

As noted previously, the deck legs, jacket members and the piles are made primarily from A36 steel. The mean strength of the A36 material was evaluated to be 43 ksi. This increase accounts for the differences between the mean and minimum specified yield strengths, and for strain rate effects associated with the wave loadings.[14,15]

Platform History. In 1969 hurricane Camille passed over the South Pass region, subjecting SP 62A and its sister platforms (SP 62B and C) to very severe wind and wave loads. Measurements and inferences from displaced equipment indicate that SP 62A was struck by both 72 ft- and 75 ft-high waves, both of which struck and inundated the lower (+45 ft MGL) deck of the platform.[16] Nearby, SP 62B was struck by a wave between 76-78 ft high, and suffered some minor damage to the lower deck and some equipment. SP 62C was loaded by a wave between 68-71 ft high based on platform damage and wave height measurements. It is remarkable in light of the fact that while each of these platforms was designed for a wave height of 58 ft, they all survived these extreme loads without any noticeable structural damage.

Analytical Model. The analytical model developed for SP 62A contained the major structural components of the platform. It was assumed the deck structural members would not fail; hence only the main framing members of the deck were included. Inelastic deformation of the deck members was suppressed. The conductor framing was included in the model, but the conductors themselves were modeled as four sets of equivalent conductors, with a diameter sized such that the Morison drag load generated on each equivalent set would be equal to the load generated on 4.5 of the original conductors. The conductor elements were declared non-structural elements so as not to contribute to the global stiffness of the platform.

Brace elements in the jacket were given a single curvature imperfection equal to 0.15 % of the brace length (L), with deformation conservatively assigned in the direction of the load. This value has been found to give good comparisons with established design buckling load vs. reduced length factor curves.[5] Imperfections were only assigned to elements for a specific load case if those elements were identified as being part of the failure mode from an initial trial analysis in which no imperfections were assigned. No imperfections were assigned to the jacket legs, as these members were expected to fail well after the majority of the bracing had failed, and hence after the collapse load had already been reached.

The analysis was performed assuming rigid joints. The joints are comprised of heavy wall branch and chord cans. Evaluation of joint capacities and stiffnesses indicated that the assumption of rigid joints was reasonable.

The main piles above the mudline were modeled discretely from the jacket sleeves, with the two constrained together for the purposes of lateral displacement. The skirt piles above the mudline were lumped together with their guides into a single element due to the fact the skirts were grouted in their guides.

To characterize the behavior of the piles below the mudline, it was decided to use the response of a single detailed pile-soil model to unit loads in order to generate a series of force-displacement relationships which could be used with the USFOS non-linear spring-to-ground element. These springs would then be attached to the ends of the piles terminating at the mudline, thus simulating appropriate foundation behavior.

A model of a single unbattered pile-soil system was developed based on the soil boring laboratory test data. Winkler spring elements representing skin friction (t-z springs), end bearing (Q-z springs) and lateral resistance (p-y springs) were generated for the static pile-soil model in accordance with API RP 2A Section 6.[11] The model was then used to generate relationships for vertical (pullout) load vs. vertical (pullout) deflection, vertical (plunging) load vs. vertical (plunging) deflection, lateral load vs. lateral deflection, and moment about the horizontal vs. rotation about the horizontal. No attempt was made to model torsional response, and no attempt was made to couple the lateral deflection and rotation about the horizontal. While the neglect of these effects might cause minor errors in the displacement of the structure, it was felt they would not significantly influence the collapse load, unless the foundation was truly the weak link (which from historical experience is believed not to be the case). The spring-to-ground element was oriented to the axis of the pile to account for batter.

After the first few trial analyses using the foundation springs based on API static criteria, the results indicated that the foundation was failing both axially and laterally. Hence, the strengths of the foundation springs were increased to recognize the biases introduced by the soil sampling, testing, analysis, and loadings.[10,11] This important issue will be discussed later in this paper.

Applied Loads. Information used to develop the loads on the platform came from several sources. The platform owner supplied information on deck dead loads and operating equipment loads, as well as projected areas for use in calculating wind and wave loads on the decks and boat landings. API RP 2A Section 17 guidelines[10] were used to develop an appropriate set of environmental loads. With the initial selection of surge/tide, current profile and wind speed profile, the wave height was adjusted until the load factor for all environmental loads was unity at the point of failure.

Based on results from oceanographic measurements storm surge and tide were selected as 3 ft.[16] Current was evaluated to be 2.3 knots, with a profile which was constant to a depth of -200 ft, varied linearly from 2.3 to 0.2 knots from -200 ft to -300 ft, and then remained constant at 0.2 knots to the mudline. Wind speed was evaluated to be 100 miles per hour (mph) for the +33 ft MGL, and at 130 mph for the +130 ft MGL (1-minute gust velocities). The period of the storm wave was evaluated to be 13.5 sec, and Stoke's V-order wave theory was used to establish the wave kinematics. In all cases, the current, wind components were those that were acting in the same direction and time as that of the wave.

Hydrodynamic coefficients were chosen based on recent studies and API guidelines. A wave kinematics factor of 0.88 was used for both the deck and jacket loads. Current blockage factors of 0.80 for broadside loads and 0.70 for end-on loads were also used. For deck wave loads, the drag coefficient C_D was chosen to be 2.5 (heavily equipped/cluttered deck). For the boat landings, C_D was chosen to be 1.2 (rough cylindrical members). One and one-half inches of marine growth was assumed from 0 ft MGL to a depth of -150 ft MGL. For cylindrical members, the drag coefficient C_D was chosen as 0.63 for smooth members, and 1.2 for rough members; the inertia coefficient C_M selected was 1.5.[13,18,20]

Analysis Cases. The platform was analyzed for three principal wave attack directions: end-on (north-traveling wave), and broadside (east- and west-traveling waves). As the piles were not expected to be the weak link, wave attack cases off the principal axes were neglected. Both broadside cases were considered as the framing to resist load in each of those directions is not anti-symmetric, as it is for the case of end-on loads. Extreme waves from the west are not expected due to the location of the structure relative to the storm track (platform to east of storm track).

In addition to varying the direction of wave attack, several different foundation fixity conditions were used. In the first set of analyses, the piles were fixed at the mudline. The results of these analyses were expected to give good estimates of both the collapse load and the failure mode (assuming failure occurred in the jacket or deck legs), and to allow the wave height which loaded the structure to collapse with a load factor of unity to be finalized. In the second and third sets of analyses, non-linear spring-to-ground elements capturing the behavior of the imbedded portions of the piles were used to represent foundation behavior. The second set analyses utilized spring-to-ground elements which were derived from the detailed pile-soil model. However, after the first few trial analyses, it was found the piles were failing both in the axial and lateral directions.

In recognition of the biases introduced by soil sampling, testing, analysis, and loadings, the strengths of the spring-to-ground elements were increased by 1.5. Post hurricane inspections of SP 62A (above and below water) did not indicate that there were any signs of significant foundation distress or displacements.[13,16] The third set of analyses made use of these stiffened and strengthened foundation springs. Results of the third set of analyses are presented and are believed to best represent the platform's ULS.

End-On Wave Attack. For the case of end-on wave attack (north-traveling wave), the wave height was adjusted until collapse of the structure occurred at a load factor of 1.05. This was achieved for an 80 ft wave, with the crest location just north of the platform's midsection. ULS was reached for a lateral load of 7440 kips. A plot of base shear vs. cellar deck displacement may be seen in Fig 2. Collapse occurred following the failure in compression of diagonal brace A3-A4 in the fourth jacket bay; this was quickly followed by the failure of the remaining compression braces in jacket bays three and four (frames A and B). It should be noted that the solution indicated minor yielding in some members before the failure of the first brace; however, it questionable as to whether any of this damage could have been ascertained during an inspection. Further, because of the rigidity of the skirt piles relative to the main

piles, indications of minor yielding in some of the horizontal and vertical framing was given as the main piles and jacket legs tended to deflect laterally more than the skirt piles and their guides. The structure exhibited extremely brittle behavior, having no reserve strength after the failure of the first brace. However, the platform still possessed approximately 75% residual strength; this was accounted for by the yielding of the jacket legs and main piles and the action of the tension braces in the third and fourth jacket bays.

Broadside Wave Attack. For the case of broadside wave attack, cases were run for both a west-traveling and an east-traveling wave, as the framing which resists loads in each direction is different. In both cases, it was found that an 84 ft wave loaded the structure to its ULS, with the crest located at the midsection of the platform.

For the west-traveling wave, collapse occurred at a load factor of 1.11, giving a total lateral load of 9510 kips. Collapse for the west-traveling wave attack case was initiated by the failure of diagonal brace A1-B1 in the second jacket bay, followed by the yielding of the tension braces in the same bay (frames 2,3,4). A plot of base shear vs. cellar deck displacement (Fig. 3) shows that the structure exhibits less-pronounced brittle behavior, and that the residual strength is quite high, on the order of 80% (following the failure of additional braces in the jacket when the cellar deck displacement is near 5 ft).

For the case of an east-traveling wave, failure was initiated by failure of diagonal braces A2-B2, A3-B3 and A4-B4 in the second jacket bay; this was followed by the yielding of the tension brace in the same bay (A1-B1). Collapse occurred for a load factor of 0.98, giving a total lateral load of 8430 kips. For this direction of wave attack, the structure exhibited nearly elastic-plastic behavior, with the collapse load being reached upon the failure of the first member. Residual strength amounted to approximately 90% of the collapse load.

Comparison with Observed Performance. SP 62A and two sister platforms were subjected to extreme environmental loads well in excess of original design loads when hurricane Camille passed through the region in 1969. The total maximum lateral loadings developed during Camille are estimated to be in the range of 6000 to 6700 kips. Based of the results of the ULS evaluation, it is apparent that SP 62A would likely have survived Camille with little or no noticeable damage. Our ULS analysis indicates the platform's static lateral loading strength is 7440 kips (end-on loading). The original design load was 3300 kips.[13] The platform's static design Reserve Strength Ratio (RSR) is thus indicated to be RSR = 2.3

Tromans and van de Graaf[13] obtained an end-on loading capacity of 8,500 kips. This capacity did not include wave-in-the-deck loadings. Based on Tromans and van de Graaf's overload ratio that accounted for deck wave loadings, an end-on loading capacity of 7230 kips was estimated. This is in good agreement with the results obtained in this study. Tromans and van de Graaf estimated that the total maximum lateral loading developed during hurricane Camille was approximately 6100 kips.[13]

The dynamic loading - nonlinear response characteristics of SP 62A have been studied.[7,8] This study resulted in an expected (best estimate) dynamic nonlinear loading capacity factor of $F_v \approx 1.2$ for this platform. These results indicate that the platform could be expected to have an ultimate dynamic lateral loading capacity of about 8930 kips or a dynamic RSR = 2.7.

Of further interest, however, is the case of SP 62B. This platform survived a large end-on wave with minor structural damage to the deck, and with no noticeable damage in the jacket.[16] Given the results of the ULS evaluation indicate an extremely brittle failure mode for this loading situation, it would be expected that there could have been some minor yielding within the jacket which would only be noticeable at a very close, detailed inspection. The results of the ULS analyses are in good agreement with the observed performance of these structures.

However, it should be recalled that earlier trial analyses based on API static pile foundation characterization guidelines indicated significant foundation displacements or failure would have occurred. Similar results were obtained by Tromans and van de Graaf.[13] No foundation damage was observed on any of the three South Pass 62 platforms. This and earlier work[1,2] clearly indicates that traditional API based static pile stiffness and capacity characterizations intended for use in design can result in significant under estimates of the pile foundation capacities and stiffnesses.

Ship Shoal 274A

Platform Characteristics. SS 274A was installed in the Ship Shoal region of the Gulf of Mexico off Louisiana in 1964. The platform is an 8-leg jacket structure sited in 213 ft of water (Fig. 4). It is an unmanned self-contained drilling and production platform, and it supports twelve 24 in.-diameter conductors. The platform was originally designed in 1963 for a 55 ft wave with no current, and Morison drag and inertia force coefficients of 0.5 and 1.5, respectively.[21] Wave forces on barge bumpers, boat landings and other appurtenances were neglected in the platform design; the original design lateral load for the platform was 1890 kips.[20,21]

SS 274A has decks at +43 ft and +57 ft MGL. The base dimensions of the jacket are 172 ft by 92 ft. The perimeter framing of the jacket is battered to 1:10. The jacket legs are fabricated from sections 46 in. in diameter (w.t. = 0.5 in., w.t. = 1.0 in. for launch truss sections in legs 2 and 3). Braces in the jacket range from 26 in. in diameter (w.t. = 0.5 in.) in the fourth jacket bay to 16 in. in diameter (w.t. = 0.5 in.) in the top bay. The major joints in the jacket are canned, with the cans having w.t. = 1.0 in.

The foundation consists of 8 piles driven through the jacket legs. The piles are 42 in. in diameter (w.t. = 0.625 in. to 1.125 in.); they pass through the legs and are welded off at +12.5 ft MGL. The deck legs are 36 in. in diameter (w.t. = 0.5

in.) and are welded to the tops of the piles. All piles were designed to have 285 ft penetration. However due to insufficient pile driver energies, most were driven to little more than 250 ft penetration. Based on on-site soil borings, the soil profile was characterized as:

0-38.5 ft	soft clay
38.5-58 ft	silty sand, fine sand
58-278 ft	stiff clay, flocculated below 140 ft
278+ ft	dense sand

It should be noted that because the piles were under-driven, they are not founded on the sand layer at 278 ft.

The deck legs, jacket members and the piles are made primarily from A36 steel. The strength of the A36 material was taken to be 43 ksi, using the same evaluations concerning material behavior that were used for SP 62A.

Platform History. In 1964, hurricane Hilda passed over the Ship Shoal region, subjecting SS 274A and nearby structures to very severe wind and wave loads.[16] Inferences from miscellaneous damage indicate that SS 274A was struck by a wave or waves in the range of 53 ft to 57 ft high. It is unclear as to whether or not the wave actually entered the lower deck.[16,20] Nevertheless, this was extremely close to the design wave height. The principal direction of the wave attack is believed to have been from the southeast.[16,20] Given the multi-directional properties of the waves close to the center of this storm, large waves could be expected to develop from wind waves from the east superimposed on swell from the south.

The platform sustained moderate damage as a result of this loading. The damage and date of discovery was as follows:

- 1964 - crack in first bay diagonal B3-A3 between corrosion wrap and horizontal. Crack in first bay diagonal B3-B2 between gussets and corrosion wrap.
- 1970 - parted bulge in leg A1 at -10 MGL. Buckled section in leg A3 at -10 MGL. Crack in first bay diagonal B2-B1 above corrosion wrap.

The damage to B3-A3 was found immediately after Hilda. The damage to B3-B2 was found three months later. The damage discovered in 1970 was found during an underwater inspection of the platform. It should be mentioned that other damage has been located since 1970, including several bent members (first bay diagonals B2-A2, A2-A3); however, it is unclear as to whether this damage occurred during Hilda or was the result of accidents or operations.[20] The damage to the platform identified during this series of inspections was repaired.

Analytical Model. The analytical model developed for SS 274A contained the major structural components of the platform. It was assumed the deck members would not fail; hence only the main framing members of the deck were included. Inelastic deformation of the deck members was suppressed. The conductor framing was included in the model, but the conductors themselves were modeled as four sets of equivalent conductors, with a diameter sized such that the Morison drag load generated on each equivalent set would be equal to the load generated on 3 of the original conductors. The conductor elements were declared non-structural elements so as not to contribute to the global stiffness of the platform.

Brace elements in the jacket were given a single curvature imperfection equal to 0.15 % of the brace length, with deformation conservatively assigned in the direction of the load. Imperfections were only assigned to elements for a specific load case if those elements were identified as being part of the failure mode from an initial trial analysis in which no imperfections were assigned.

The analysis was performed assuming rigid joints. The platform joints are comprised of heavy wall chord and branch cans. Evaluation of joint capacities and stiffnesses indicated that the assumption of rigid joints was valid.

The main piles above the mudline were modeled discretely from the jacket sleeves, with the two constrained together for the purposes of lateral displacement. To capture the behavior of the piles below the mudline, the same approach was used as for the analysis of SP 62A. The initial foundation characterization was based on the laboratory soil test data from the site soil boring and API static pile guidelines. After the first few trial analyses using these foundation springs, results indicated that the foundation was failing laterally; lateral displacements at the mudline were excessive. Hence, the strengths and stiffnesses of the foundation spring models were increased to recognize the effects of site soil sampling, testing, and dynamic - cyclic loading effects. This issue will be discussed later in this paper.

Applied Loads. Information used to develop the loads on the platform came from several sources. The platform operator supplied information on deck dead loads and operating equipment loads, as well as projected areas for use in calculating wind and wave loads on the decks and boat landings. API RP 2A Section 17 guidelines were used to develop an appropriate set of environmental loads.[10] With the initial selection of surge/tide, current profile and wind speed profile, the wave height was adjusted until the load factor for all environmental loads was unity at the point of failure.

Based on results from oceanographic studies, surge/tide was selected as 3 ft. Current was estimated to be 2.3 knots, with a profile which was constant to a depth of -200 ft, varied linearly from 2.3 to 0.2 knots from -200 ft to -300 ft (cutoff at the mudline, -213 ft). Average wind speed was estimated to be 125 mph for the decks (1-minute gust velocities). The period of the storm wave was estimated to be 13.5 sec, and Stoke's fifth order wave theory was used to determine the wave kinematics. In all cases, the current and wind were considered to be acting in the same direction and at the same time as the wave.

Hydrodynamic coefficients were chosen based on recent studies and API guidelines. A wave kinematics factor of 0.88 was used for both the deck and jacket loads. Current blockage factors of 0.80 for broadside loads and 0.70 for end-on loads were also used. For deck wave loads, the drag coefficient C_D was chosen to be 2.5 (heavily equipped/cluttered deck). For the

boat landings, C_D was chosen to be 1.2 (rough cylindrical members). One and one-half inches of marine growth was assumed from 0 ft MGL to a depth of -150 ft MGL. For cylindrical members, the drag coefficient C_D was chosen as 0.63 for smooth members, and 1.2 for rough members; the inertia coefficient C_M selected was 1.5.[13,18,20]

Analysis Cases. The platform was analyzed for two principal wave attack directions: end-on (north-traveling wave), and broadside (west-traveling wave). As the piles were not expected to be the weak link, wave attack cases off the principle axes were neglected.

In addition to varying the direction of wave attack, several different foundation fixity conditions were used. In the first set of analyses, the piles were fixed at the mudline. The results of these analyses were expected to give good estimates of both the collapse load and the failure mode (assuming failure occurred in the jacket or deck legs), and to allow the wave height which loaded the structure to collapse with a load factor of unity to be finalized. In the second and third sets of analyses, non-linear spring-to-ground elements capturing the behavior of the imbedded portions of the piles were used to represent foundation behavior; these were derived as described in the discussion on platform modeling. The second set analyses utilized spring-to-ground elements which were derived from the detailed pile-soil model based on static pile - soil interaction characterizations. However, after the first few trial analyses, it was found the piles were failing both in the axial and lateral directions.

In recognition of soil sampling, testing, analysis and dynamic loading effects, the lateral strengths of the spring-to-ground elements were factored up by 1.5. The lateral stiffnesses of the springs were increased by a factor of 4 to recognize the foregoing soil - pile interaction effects. Platform inspections following Hilda did not disclose the presence of any significant foundation deformations. The third set of analyses made use of these factored foundation springs. The results of this third set of analyses are presented as best characterizing the platform's ULS.

End-On Wave Attack. For the case of end-on wave attack (north-traveling wave), the wave height was adjusted until collapse of the structure occurred at a load factor of 0.957. This was achieved for a 67 ft wave, with the crest location just north of the platform's midsection. ULS was reached for a lateral load of 4750 kips. A plot of base shear vs. cellar deck displacement is given in Fig. 5. Collapse occurs following the failure in compression of diagonal brace B3-B4 in the third jacket bay; this was quickly followed by the failure of the remaining compression braces in jacket bays three and four (frames A and B). The structure exhibits extremely brittle behavior, having no reserve strength after the failure of the first brace.

Broadside Wave Attack. For the case of broadside wave attack (west-traveling wave), the wave height was adjusted until collapse of the structure occurred at a load factor of 0.99. This was achieved for a 67 ft wave, with the crest location at the platform's midsection. ULS was reached for a lateral load of 5250 kips. A plot of base shear vs. cellar deck displacement is given in Fig. 6. The collapse mechanism involves several members. The first member to fail is diagonal brace A4-B4 in the third jacket bay; this member fails in compression. This failure is followed by the failures of diagonal braces A2-B2 (first jacket bay) and A3-B3 (second jacket bay), both in compression. It should be noticed there is a slight amount of reserve strength following the failure of A4-B4. It should be noted that the solution indicated yielding in other members prior to the initiation of collapse.

Comparison with Observed Performance. As noted previously, Ship Shoal 274A was subjected to extreme environmental loads well in excess of original design loads when hurricane Hilda passed through the region in 1969. The maximum lateral loadings developed during Hilda were estimated to be 3500 kips to 4000 kips. Van de Graaf and Tromans estimated the maximum total lateral loadings developed during hurricane Hilda to be 3600 kips.[20] The original design lateral load for the platform was 1890 kips.[20,21]

The results of our ULS evaluation indicate that SS 274A would likely have survived Hilda with little or no damage (end-on lateral static loading capacity of 4750 kips; static design RSR is thus 2.5). Consideration of the dynamic loading - nonlinear response characteristics of SS 274A indicate an expected dynamic nonlinear loading capacity factor of $F_v \approx 1.1$ for this platform.[7,8] These results indicate that the platform could be expected to have an ultimate dynamic lateral loading capacity of about 5200 kips. The platform has a dynamic RSR = 2.8, comparable with that of SP 62 A.

The analyses performed by van de Graaf and Tromans indicated that this platform had a total lateral loading capacity of approximately 4800 kips when the foundation failure modes were suppressed.[20] These results are in excellent agreement with those developed here. In addition, as was found during these analyses, van de Graaf and Tromans found that conventional static pile stiffness and capacity characterizations resulted in unreasonable results.[20] Their results indicated that the platform would fail in the foundation (laterally and axially) at the maximum loadings estimated to have occurred during hurricane Hilda. These failures occurred before there were any failures in the jacket. This result was not in agreement with the results of the platform inspections that were performed following Hilda. Lateral and axial pile failure had to be suppressed before reasonable results could be obtained.

The exact nature of the damage recorded following Hilda is not readily evident based on the results of the ULS analyses performed during this study. Consequently, a more detailed examination of the results was developed. A summary of this development follows.

If the damage to brace A2-B2 occurred during Hilda, this would be confirmed by the analysis results. However, it is unclear as to whether this is the result of a separate event; van de Graaf and Tromans, 1991, assumed the damage was due to the loadings developed during hurricane Hilda.[20]

The failures noted in first bay braces A3-B3, B2-B3 and B1-B2 are most likely due to low cycle fatigue failure and fracture. In each case, the members suffered cracking just above the corrosion wrap near the brace end. This configuration of wrapped members and lapped brace joints stiffened by gussets is notorious for resulting in extremely high stress concentrations conducive to crack formation. Current API guidelines recommend a stress concentration factor of no less than 6 for lapped configurations.[11] Stress values at the brace ends which cracked for the three first bay diagonal braces are listed below, along with the load cases corresponding to the state of stress:

A3-B3 - 25 ksi - Broadside LF=0.85

B2-B3 -15 ksi - End-On LF=0.85

B1-B2 -13 ksi - End-On LF=0.85

The reduced load factor was used to estimate the maximum stresses associated with hurricane Hilda and to reflect the fact that the load at which these failures occurred was not due to the extreme (67 ft) wave used in the study. Given a likely stress concentration factor in the range from 5-10, and assuming 10 significant stress cycles, it would appear A3-B3 would stand an excellent chance of initiating a crack during an extreme loading event such as Hilda. B2-B3 and B1-B2 seem less likely to suffer cracking, but the chances of such an event are still reasonable.

For the damage to the upper portions of legs A1 and A3, it is apparent there was local buckling of the leg . Stresses in each leg segment at -10 ft MGL are shown below, along with the load case used:

A1 -28 ksi - Broadside LF=0.85

A3 -28 ksi - Broadside,LF=0.85

Local buckling is a complicated phenomenon which is quite sensitive to material imperfection and member eccentricities[17], consequently it is rather difficult to predict. API guidelines (Section 3.2.2)[11] indicate stresses should not exceed 33 ksi for 36 ksi steel sections with D/t=92; using the assumed yield of 43 ksi and following API-recommended inelastic local buckling guidelines would mean an upper limit of 40 ksi for local buckling prevention with this same D/t ratio. Given the moderate diameter-to-thickness ratio of the legs at this location, coupled with the presence of the corrosion wrap above and a joint can below each section, it seems possible that local buckling would indeed take place.

Overall, the results of the ULS evaluation are a good reflection of the lateral load capacity and performance of SS 274A. The types of local failures which occurred normally will not be exposed during a static pushover evaluation of the type performed during this study. Even if care is taken to perform detailed checking of local failure phenomenon, there are many uncertainties which make the prediction and assessment of local failures very difficult. This should be taken as a warning that a ULS evaluation of the type performed will not always catch all possible modes of damage and/or failure which may occur; hence, good engineering judgment will be needed in order to identify problem areas a computer simulation may ignore.

Parametric Analyses

In addition to performing the ULS verification studies, several areas of the ULS analysis process were studied parametrically. Two of these areas, that of modeling of foundation piles and the effects of member imperfection on the ULS, have already been addressed by Loch and Bea.[2] The third area, that of determining the effects of horizontal framing on the ULS, is of importance with regards to a key assumption in the program ULSLEA[1], which is that the presence or lack of horizontal framing above and below each jacket bay in a template-type structure does not affect first member failure.

Modeling Foundation Piles. Difficulties in modeling the behavior of foundation piles have been well documented in numerous sources as summarized by Loch and Bea.[2] A summary of the magnitudes and sources of the mean biases (expected value / nominal value) is given in Table 1 for steel piles driven into cohesive soils.

During the performance of the ULS analyses of SP 62A and SS 274A, preliminary results based on API static pile lateral and axial characteristics indicated that the pile foundations were failing in both platforms. In light of historical data on both structures, it was judged this was unrealistic. Hence, modifications were made to the foundation elements to recognize site soil sampling, testing, analysis, and dynamic - cyclic loading effects (Table 1).

For SP 62A, the initial analyses indicated that the foundation piles were failing through plunging and pullout. This was most evident for the case of broadside wave attack. In recognition of soil sampling, testing, analysis and dynamic - cyclic loading effects, the foundation elements had their axial strengths factored up by 50 %. A plot of global load vs. displacement for the analysis using the modified foundation is compared with results from the unmodified foundation elements in Fig. 7. The results indicate that while the displacement history of the platform has changed substantially, the peak load capacity of the platform has not changed significantly.

For SS 274A, the initial analyses indicated that the structure was undergoing unrealistically large lateral displacements at the mudline. This was due in part to the fact the foundation was failing laterally, but also because the stiffness of the foundation elements was very low. Hence, in recognition of the soil sampling, testing, analysis, and dynamic - cyclic loading effects (Table 1) the foundation elements lateral capacities were factored up by 50 %. In addition, the stiffness of the elements were increased by a factor of 4. Plots of global load vs. cellar deck displacement for SS 274A using both sets of foundation elements are summarized in Fig. 8. Again, it should be noted

that while the displacement behavior is much more reasonable, the overall load capacity of the platform has not changed significantly.

These two incidents served to highlight the difficulties associated with accurately modeling foundation behavior. Soil sampling and testing procedures, analytical procedures, and dynamic - cyclic loading effects on pile behavior all contribute to the difficulties associated with modeling a foundation realistically.

Diagonal Brace Imperfections. Diagonal bracing in jacket structures is designed to primarily resist axial load. Hence, the strength of a diagonal brace can be characterized by both its strength in tension and its stability in compression.

USFOS allows a user to input an initial imperfection to members in order to account for their out-of-straightness and/or general buckling performance calibration. The effects of imperfection size and orientation were explored in some detail by Loch and Bea.[2] As part of a continuation of this effort, it was decided to study the effects of varying critical member orientation and size for an analysis of SP 62A.

As mentioned previously, initial member imperfection was set at 0.15% of the brace length (L), with the imperfection in the direction of the applied load. Over a series of analyses utilizing the end-on wave attack load pattern, member imperfection was varied from +0.3% L (with the load direction) to -0.3% L (against the load direction). The results of this variation on the load factor at collapse are summarized in Fig. 9.

Keeping in mind the original load factor for SP 62A at collapse was 1.05, it can be seen that varying the member imperfection and orientation can change the resulting collapse load by at much as 15%. This is a very significant amount; it should be clear that simply increasing imperfection size without paying heed to the orientation of the imperfection may produce analytical results which are unconservative.

Effect of Horizontal Framing. Of particular interest to the development of ULSLEA is determining the effects of horizontal bracing on the ULS of steel template-type platforms.[1] A fundamental assumption the ULSLEA program uses when evaluating the strength of diagonal braces in jacket bays is that each bay is supported by rigid framing top and bottom, regardless of whether there is actually sturdy horizontal framing at each level. It is believed the presence or lack of this framing will not influence first member failure. Hence, each jacket bay is essentially evaluated as a braced portal frame.

To evaluate the validity of the assumption used by ULSLEA, a study was performed utilizing the model of SP 62A. SP 62A does not possess horizontal framing at every level, hence, it was decided to add horizontal members at each level between the jacket bays, and then study the effects on the ULS.

Global results for the SP 62A are plotted in Fig. 10 (end-on) and Fig. 11 (broadside). For the case of end-on wave attack, the structure's ultimate load capacity has not changed, but the amount of residual strength has increased. This is due to the fact that the tension braces in the bays above and below the failed compression braces have taken up the load shed by the failed braces more effectively with the presence of the horizontal framing members.

For the case of broadside wave attack (east-traveling wave), the effect of the bracing is negligible. There has been a slight increase in reserve strength, and slight increases in residual strength. The most dramatic change is seen in the case of broadside wave attack (west-traveling wave). There has been a 20% increase in ultimate capacity beyond first member failure, and the platform exhibits much higher reserve strength.

These results indicate that the assumption used in ULSLEA is valid. While the presence of strong horizontal members may aid in load redistribution following first member failure, it does not appear to affect first member failure, which is what ULSLEA uses to characterize platform capacity. Nevertheless, this effect should be studied further. The results from the broadside wave attack (west-traveling wave) indicate that substantial strength beyond first member failure may be obtained by providing a horizontal framing system by which load can be redistributed; this effect should not be ignored when considering methods of increasing the overall robustness and ductility of template-type structures.

Conclusions and Observations

ULS Assessments. The ULS evaluations of South Pass 62A and Ship Shoal 274A indicate both platforms possess strength substantially in excess of their original design loads. For SP 62A, it was found that end-on (north-traveling) wave attack governed the ULS. The maximum static lateral load capacity was estimated to be 7440 kips. The maximum dynamic lateral load capacity was estimated to be 8930 kips. The original design load for SP 62A was 3300 kips. This gives a ratio of 2.3 to 2.7 between the static and dynamic ULS capacity and the design load, respectively.

For SS 274A, it was found that end-on (north-traveling) wave attack governed the ULS; the maximum static lateral load capacity was estimated to be 4750 kips. The maximum dynamic lateral load capacity was estimated to be 5200 kips. The original design load for SS 274A was 1890 kips. This gives a ratio of 2.5 to 2.8 between the static and dynamic ULS capacity and the design load, respectively.

Overall, the ULS evaluations of South Pass 62A and Ship Shoal 274A indicate the validity of using detailed static pushover analyses in studying the performance of steel template-type structures subjected to hurricane wind and wave loads. The analyses indicate survival was likely for both platforms during their respective severe hurricane loadings. The results from these analyses are in agreement with the observed global performance of the two structures in the hurricanes.

However, the failures that were found in SS 274A following hurricane Hilda did not manifest themselves at the level of detail initially considered in the analyses. It took a more detailed evaluation of the forces acting on the jacket members

and the stresses induced in the members in order to determine the possibility of such failures occurring, and indeed, it is conjectural as to whether these failures could have actually been predicted with any sort of certainty. While detailed static pushover analysis of the type described herein is certainly a useful tool for the evaluation of structural performance, it must be applied carefully by experienced engineers in order to ensure atypical or unexpected failure modes are considered.

General Observations. In both cases, the initial analyses were based on conventional design basis foundation pile - soil interaction characteristics, and in both cases, these characteristics resulted in unrealistic performance characteristics of the platforms. The platforms were predicted to fail when they did not and to fail in a manner that was not consistent with post hurricane inspections. For the purposes of platform reassessment studies, more realistic characterizations of the soil parameters need to be developed and documented.

Accounting for expected mean values of yield strength and strain-rate effects has increased the estimated capacities of each structure by approximately 15-20 %. Consideration of the calibration of the brace buckling characteristics (by adjusting the initial single-curvature imperfection) was seen to have the effect of biasing the results by as much as 15 %, and if care was not taken in modeling this could be applied unwittingly in an unconservative fashion. Consideration of dynamic loading effects could reveal load reduction (or capacity increase) on the order of an additional 10-20 %. These effects, when taken together, may give an aging platform an additional margin of strength which should be considered by the analyst when faced with the task of structural reassessment. It is essential that identified biases and effects such as these be taken into account if realistic ULS performance estimates are to be obtained.

Lastly, it was established that the presence of additional horizontal framing has little effect on first member failure in the jacket; this may be taken as initial validation for the program ULSLEA[1], which assumes platform capacity is governed by first member failure.

Acknowledgments

This project was sponsored by the U.S. Minerals Management Service in cooperation with the California State Lands Commission. The parallel project to develop and verify ULSLEA was sponsored by ARCO Exploration and Production Technology, Exxon Production Research Company, Mobil Technology Company, Shell Offshore Incorporated, and Unocal Corporation.

Appreciation is expressed to SINTEF for use of USFOS and to DNV for use of PREFRAME and WAJAC in this research.

Special appreciation is expressed to Shell Offshore Incorporated, Mr. Kris Digre and Dr. Rabi De, for the data and information on SP 62A and SS 274A.

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TABLE 1 - PILE AXIAL LOADING BIASES*
Bias = True Pile Top Capacity / Computed Pile Top Capacity

Factor	Done with	But could be done with	Ref. bias	Bias
Drilling	sea water	with mud	1.0	1.5
Sampling	wireline	push	1.0	1.5
Preserve	extrude, wax	shelby tubes	1.0	1.2
Testing	unconfined	direct shear	1.0	2.0
Loading	static	wave, quake	1.0	1.5
Loading	1 cycle	100 cycles	1.0	0.8
Age	10 days	10,000 days	1.0	1.5
Analysis	limit equilibrium	nonlinear finite elmt.	1.0	0.9

*ranges for axial loadings, 4-6 ft diameter steel piles driven 300 to 400 feet into normally consolidated cohesive soils in water depths of 200 to 300 feet, designed according to API Limit Equilibrium methods and factors of safety. Maximum bias including all factors = 8.75. Old boring and design conditions bias = 3 to 4. State of the art (1996) bias on static capacity = 1.2 to 1.4

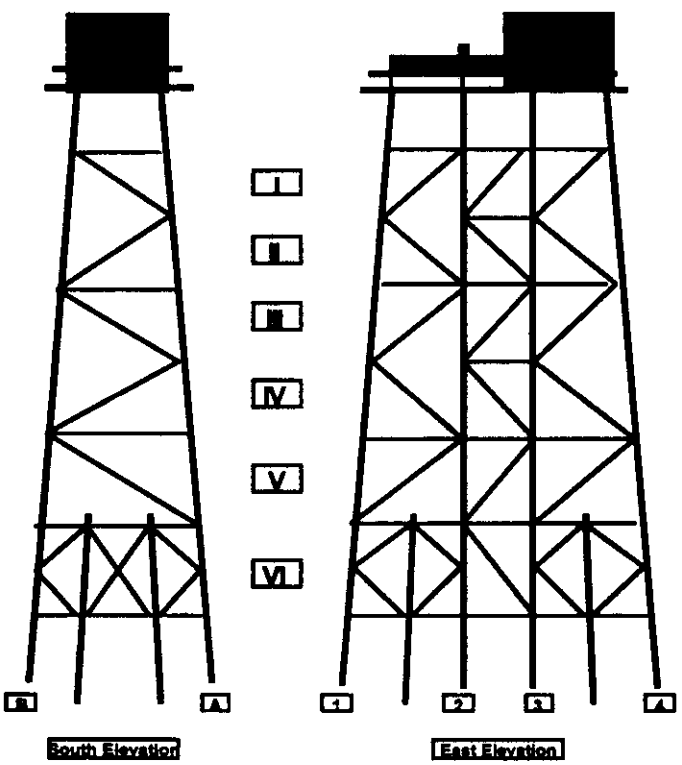

Fig. 1 - SP 62A platform elevations

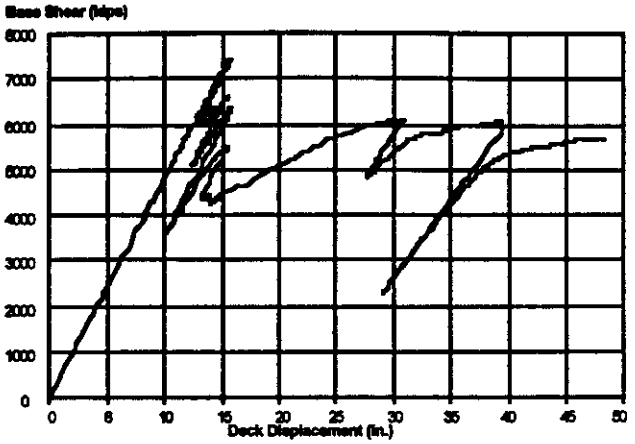

Fig. 2 - Base shear vs. cellar deck displacement
(80 ft wave end-on from south)

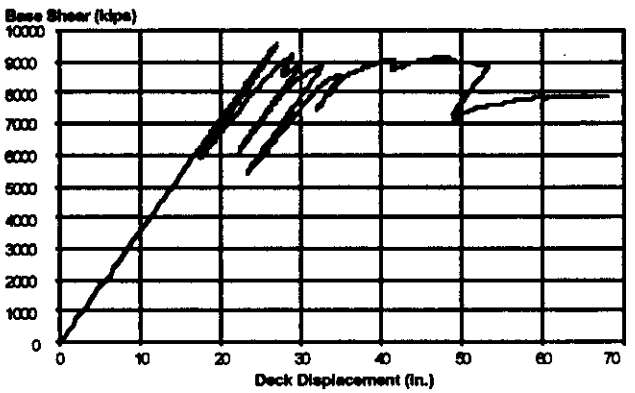

Fig. 3 - Base shear vs. cellar deck displacement
(84 ft wave broadside from east)

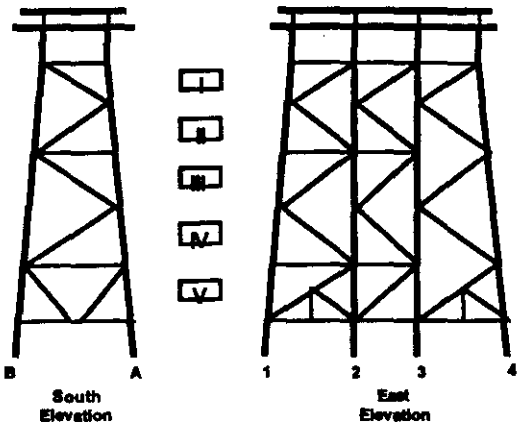

Fig. 4 - SS 274A platform elevation

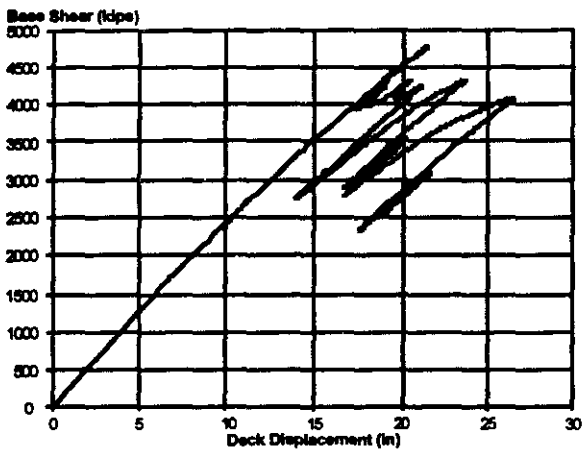

Fig. 5 - Base shear vs. cellar deck displacement
(67 ft wave end-on from south)

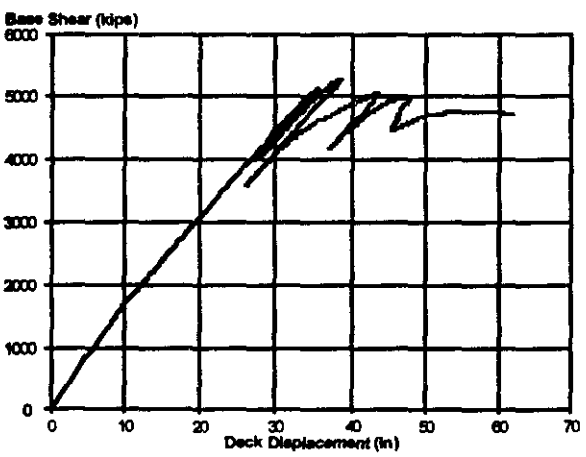

Fig. 6 - Base shear vs. cellar deck displacement
(67 ft wave broadside from east)

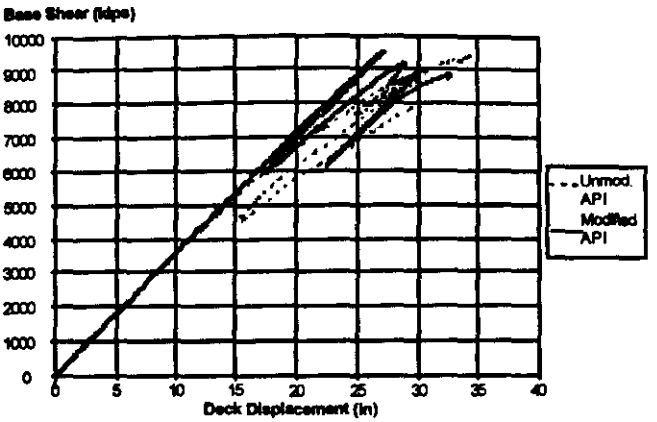

Figure 7 - Effects of modified and unmodified foundations on global behavior of SP 62A

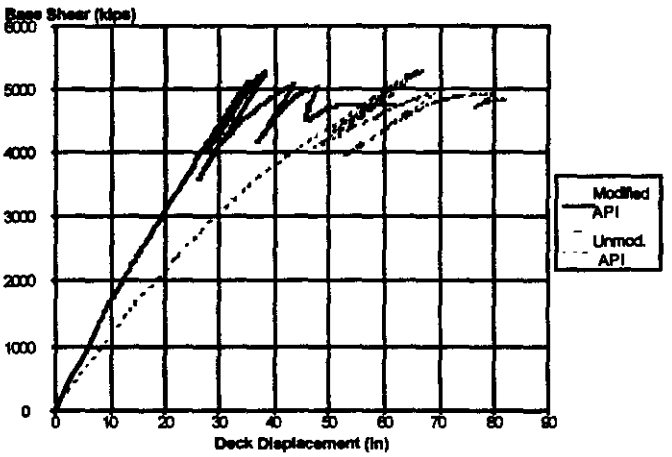

Fig. 8 - Effects of modified and unmodified foundations on global behavior of SS 274A

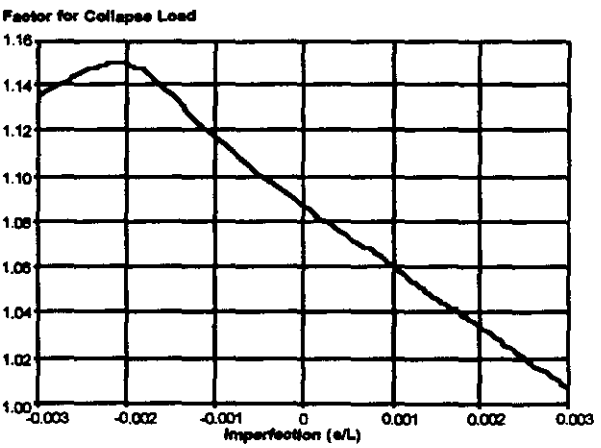

Fig. 9 - Effect of brace imperfection on load factor
at collapse

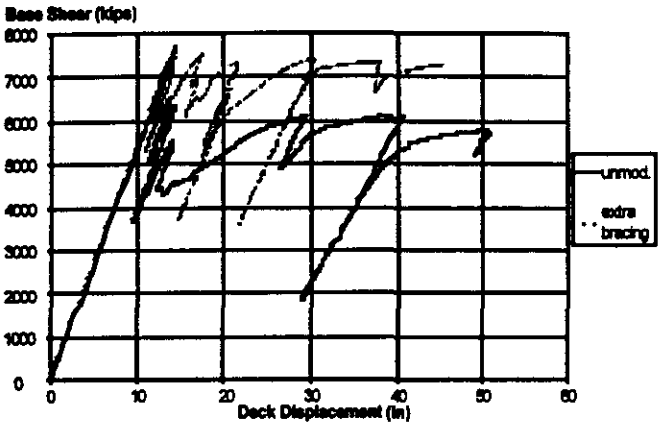

Fig. 10 - Effect of additional horizontal framing on SP 62A (80 ft wave end-on from south)

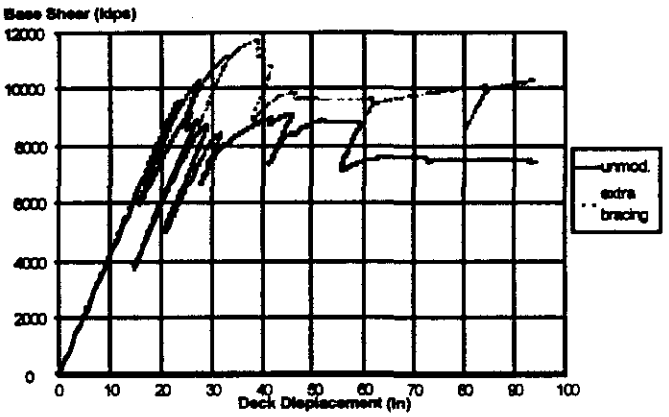

Fig. 11 - Effect of additional horizontal framing on SP 62A (84 ft wave broadside from east)