

Effect of Best-estimate Geotechnical p-y Curves on Performance of Offshore Structures

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Executive Summary

Fixed offshore platform pile foundation design and assessment in the Gulf of Mexico (GOM) has been based on API RP 2A (various editions) and API/ISO RP 2GEO, 1st Edition. Over the years, post-hurricane observations of the platform performance in the GOM have shown that these standards tend to underestimate soil stiffness and capacity. This underprediction does not necessarily lead to conservative structural design and assessment. For example, fatigue design requires a realistic estimate of structural dynamics and strength design with a clear definition of structural load path and failure mechanism.

Research work, consisting of small- and large-scale pile tests, laboratory testing, and numerical analyses, has been carried out for years to understand this conservatism and define best-estimate geotechnical criteria, i.e. best-estimate soil lateral reaction for pile foundation design. These efforts culminated in the proposed draft of API/ISO RP 2GEO, 2nd Edition, which systematically re-defines the clay soil lateral reaction formulations for piles under monotonic, cyclic, and fatigue loads.

The goal of this study, sponsored by API SC2, was to perform structural analyses using soil models developed by 2GEO, 1st Edition criteria and 2GEO, 2nd Edition draft criteria to determine the effect of the new clay soil p-y formulations on the structural responses of these platforms. The predicted responses were then compared with measured or observed platform performance in field.

Three (3) featured fixed offshore platforms in the GOM, i.e. a free-standing caisson structure, a jacket type platform, and a tripod platform, were selected as representative structures for the study. The selected platforms are actual platforms in operation in the GOM. The caisson structure and jacket platform were damaged in Hurricanes Andrew and Ike, respectively. Platform vibration of the tripod platform was measured and recorded in the field to verify the tripod structural integrity. All relevant platform design, operation, and maintenance data of these structures are sufficient and available for the study.

Results of the analyses performed in this study demonstrate that structural models using the soil reaction formulations of the proposed 2GEO, 2nd Edition (draft), in combination with the best-estimate soil profiles interpreted based on the standard direct simple shear (DSS) tests, adequately predicted the platform responses observed in hurricanes and in normal sea conditions.

The clay soil p-y reaction formulations (monotonic, cyclic, and fatigue), when used with best-estimate DSS shear strength profiles as proposed in the draft of 2GEO, 2nd Edition, show significant improvement over the formulations in 2GEO, 1st Edition and older RP 2A editions (e.g. the so-called “Matlock curves”), and are endorsed and recommended for pile foundation design and assessment. In particular, the use of the new cyclic p-y curves is recommended for platform structural analysis under extreme environmental conditions.

1 Introduction

1.1 Background

Post-hurricane observations of fixed offshore platform performance in the Gulf of Mexico (GOM) often demonstrate platform structural damage or failure rather than foundation failure. However, platform analysis and assessment by API RP 2A (various editions) and API/ISO RP 2GEO, 1st Edition (Ref. 1, 2) foundation design approach are frequently governed by the foundation capacity and predict failure in the piles rather than the jacket structure.

Subsea wellhead and conductor fatigue design and analysis also predict unrealistic fatigue life expectancy due to the deviated dynamics caused by underestimated soil stiffness near the mudline. These have led to questions of unknown reliability and level of conservatism in the current API recommended pile foundation design approach.

Unrealistic conservatism in pile foundation design may have significant impact on new platform design and existing platform assessment. For new platform design, it may lead to excessively larger, deeper pile design associated with higher cost. Fatigue life can also be miscalculated due to inadequate estimates of platform dynamics (e.g. natural periods). For existing platform assessment, it may lead to misjudgment of platform actual capacity (due to pre-mature foundation failure in analysis) and locations of critical structural components (for platform repair and strengthening).

Research work and studies on fixed offshore platform performance in GOM hurricanes, funded by MMS/BSEE, API, and industries, have been carried out for decades (Ref. 5-12, 14-15, 23-26). A general perception indicates that pile foundations appear to be stronger and stiffer than calculated. Pushover analysis for jacket structures to their ultimate capacities commonly indicates that the foundation governs the capacity of the structural system. However, there are few, if any, observed pile foundation failures that have occurred in these hurricanes.

Discrepancy between predicted and observed performance of platform foundation system may be in part due to:

- underestimation of soil stiffness and capacity on the soil-structure interaction curves, especially the pile foundation soil lateral p-y curves for clays;
- interpretation of geotechnical soil boring and test data;
- loading rate and cyclic effect on pile capacity under short term wave loading conditions;
- aging effect of increased pile capacity for years or decades after installation;
- platform computer modeling inaccuracies and over-simplifications;
- disproportionate redundancies associated with structure components and foundation system in existing design practice;
- incomplete or misguided structural data and information.

Geotechnical research work, consisting of small- and large-scale pile tests, laboratory testing, and numerical analyses, has been performed for years to check the pile foundation design and define the best-estimate geotechnical criteria (Ref. 16-18, 20, 29-33). This culminated in the proposed draft of API/ISO RP 2GEO, 2nd Edition (Ref. 3, Annex D), which systematically re-defined the soil lateral reaction formulations, especially for clays.

1.2 Objectives

This study, functioning as an engineering practice and exercise, is to perform structural analysis and assessment on real GOM platforms, using field collected data, detailed structural modeling and advanced analysis techniques, to test the best-estimate geotechnical criteria based on the proposed 2GEO update (Annex D). The structural analyses performed are mainly in the form of nonlinear dynamic pushover analysis.

The objectives of this study include:

- Compare the effect of different geotechnical criteria (current design API RP 2A and 2GEO, 1st Edition vs. proposed 2GEO, 2nd Edition draft) on platform performance;
- Compare analytical model prediction to field platform performance and measurement;
- Determine and offer best-estimate geotechnical criteria based on these;
- Identify issues related to design application.

The study includes extensive work of literature review, structure identification and selection, platform field data collection and comparison, and required structural modeling and analytical work.

This study mainly focuses on analysis of fixed offshore platforms to compare with their field performance. Studies on subsea wellhead/conductor system have been performed by others (Ref. 13, 19, 22, 27, 28, 30, 31) and are not covered in the scope of this study.

1.3 Platform Selection

The goal of this study is to perform a direct comparison of the structure field performance and analytical prediction. Selection of the candidate offshore platforms is key to the following work. The platform selection criteria include the following major aspects:

- Availability of reliable and relevant site-specific soil data (all-clay sites preferred);
- Availability of platform historical design, installation, operation, and maintenance data, such as drawings, models, photos, topsides weight and platform condition at storm time, platform historical inspection reports, etc.;
- Platform performance in large storms/hurricanes, including damage or failure condition, pre- and post-storm inspection data etc.;
- Storm/hurricane hindcast metocean data at specific sites;
- Platform monitoring or field measurement data, such as deck acceleration and vibration recording;
- Structures with minimal structural complexity (such as a caisson) preferred.

Candidate structures were screened and identified from previous research studies (Ref. 5-9, 14-15, 23-26) and industry projects. Structural data and information were collected, and preliminary assessment was carried out to finalize the selection to following three (3) structures in the GOM, for detailed study:

- SS-Caisson. This is a single column caisson structure, damaged in Hurricane Andrew;
- SS-Platform. A jacket type platform, damaged in Hurricane Ike;

- EI-Tripod. A tripod structure with platform natural frequency measurement data, survived Hurricanes Katrina, Rita, and Ike with no major structural damage.

Although these platforms have been anonymized for confidentiality reasons, their structural data and information are all based on real platform data, historical records, and structural performance in hurricanes.

1.4 Structural Models and Analysis Techniques

The structure computer models were developed in CAP (Capacity Analysis Program, CAPFOS Inc.) using available platform data, information, and computer models.

CAP is an advanced offshore structural analysis program developed for nonlinear static/dynamic, pushover/failure analysis under the effect of wave, earthquake, and wind loadings. Compared to other industry software packages, such as SACS and Sesam/Usfos, CAP provides more options for special analysis elements, especially for pile-soil interaction, and modeling techniques tailored for more advanced users.

This study requires a significant amount of analytical and computational work to assess the platform performance and predict its field response in large storms. This involves fully nonlinear structural analysis to consider nonlinear soil springs (p-y, t-z, Q-z springs), dynamic wave effects, tubular member yielding and buckling, tubular joint damage and failure, and structural collapse with large deformation etc. The structural models (member elements and tubular joints) developed in this study are capable of capturing these damage and failure modes, if occurred.

The structural models developed are based on the best-estimate structural data, including best-estimate geotechnical criteria and best-estimate structural details. Many structural details and factors may potentially affect the analysis results. Significant effort has been made to extract correct data and accurately model the structures to their best-estimate “as-was” conditions during the time of the concerned events. This is to single out the effect of soil models from other structural details and factors, and to focus on the study objectives.

Pushover analysis, in a static way as we generally imply, is a standard analysis technique to assess the structural ultimate capacity. In pushover analysis, the lateral environmental loadings on the structure are statically increased until the structure collapses, which also assumes a quasi-static structural response. This is generally conservative and acceptable from a design and assessment point, but does not fully reflect the actual cycles of structure dynamic reactions and wave responses in hurricanes. This limitation should be noted when using the static pushover analysis.

In this study, an explicit dynamic pushover analysis technique was adopted to more accurately simulate the dynamic wave effect on structures. This dynamic pushover analysis is to run a wave train (either regular wave or random wave) through the structure dynamically in the time domain to monitor the structure dynamic response and actual performance. For this specific study, this approach is considered a more realistic and better approach than the static pushover, though computational complexity increases.

Other special structural modeling techniques have also been used in the study for specific reasons. Although this study focuses on the best-estimate geotechnical criteria, a lesson learned is that selection of appropriate structural modeling techniques and analysis methods is also important and should be properly addressed in engineering practice.

1.5 Project Team

This study was funded/supported by API Subcommittee 2 (SC2) on Offshore Structures and its resource groups. Energo Engineering was the main interface and prime contractor for the study, and performed most project related data gathering, analysis and reporting work.

2 API/ISO Pile Foundation Soil Reaction Curve Update

The soil-structure interaction curves for lateral soil resistance to piles and conductors (i.e. p-y curves), as currently formulated in API/ISO RP 2A and 2GEO, 1st Edition (Ref. 1, 2), tend to underestimate soil stiffness and capacity for clays. This can result in underestimation of fatigue life and structural foundation capacity for fixed offshore structures and floating platform riser/wellhead systems.

The current API p-y criteria for marine clay were based on Matlock (Ref. 21) and originally intended for design of shallow water jacket pile foundations. The p-y curves tend to underestimate both initial stiffness and ultimate resistance of lateral soil response for clays. Several p-y criteria were developed recently to improve the prediction of structural performance for specific applications. For example, Jeanjean (Ref. 18) proposed the monotonic p-y curves in clays and Zhang (Ref. 32, 33) proposed the corresponding cyclic p-y curves. Zakeri (Ref. 30, 31) developed the applicable p-y curves for fatigue assessment of subsea riser/wellhead/conductor systems. Additionally, the Jeanjean p-y curves (Ref. 16) were demonstrated to provide a much-improved agreement on the dynamic responses of a jacket structure under earthquake excitations than the current design API RP 2A and 2GEO, 1st Edition p-y criteria in a recent seismic soil-structure interaction centrifuge test by Litton (Ref. 20).

Post-hurricane observations of fixed platform performance in the GOM have indicated that the pile foundation capacities are higher than those established by analysis (Ref. 6-9, 14, 15, 23-26). Subsea wellhead and conductor fatigue monitoring and analysis also predict unrealistic fatigue life due to the effect of weak soil stiffness near the mudline (Ref. 13, 19, 22, 27, 28).

A review of the historical studies on post-hurricane observations suggest that a validation of the new clay soil p-y curve formulations and an update to industry recommended practice is required. Updating the p-y curve formulation for clays, as proposed in the draft version of API RP 2GEO, 2nd Edition (ISO 19901-4 3rd Edition) (Ref. 3, Annex D), may significantly improve the industry design and practice of fixed offshore structures and floating systems riser/wellhead systems.

3 Assessment of Leaning Caisson in Hurricane Andrew

3.1 General

SS-Caisson is a single column caisson type of platform in the West-Central region of the GOM. The platform was damaged with a reported 15-degree tilt during Hurricane Andrew in 1992 (Ref. 5, 24, 25).

3.2 Caisson Structure Detail

The SS-Caisson is a free-standing wellhead platform in 53 ft water depth installed in 1983 in GOM. The original design drawings showed a 48 ft water depth, whereas the actual water depth at the caisson location was reported as 53 ft, which is determined to be the correct water depth.

The caisson was damaged during Hurricane Andrew in 1992 with a 15-degree leaning angle at waterline. No other structural damage was observed and reported, and it was suspected that the leaning was caused by foundation failure below the mudline. Figure 3.1 shows a similar caisson platform leaning damage in hurricanes.

Figure 3.2 provides the structural details of the caisson design. The major platform characteristics are:

- Caisson installed in 1983;
- Water depth 53 ft;
- Caisson & pile tubular 48"Ø 1.0"~2.0" (diameter and wall thickness);

- Marine growth thickness 1.5", based on API recommendation;
- Caisson material A36 steel;
- Caisson soil penetration 95 ft, site soil—soft clay;
- Topsides main deck at EL(+) 14'-6" above MLW level, deck size ~ 25'x14';
- Topsides total weight approximately 30 kips (steel+equipment+inner_string_tension);
- Caisson damaged during Hurricane Andrew in August 1992, leaning 15 degree at waterline.

The caisson has multiple inner production casings. They are not grouted in the annuli over the length of the caisson. The diameters of inner casing strings are small (7-10") as compared with the caisson diameter, therefore their string tension effect on caisson compression buckling capacity is negligible. Only the caisson tubular is considered the main structural component to contribute to structural strength.

For structural ultimate capacity assessment, the mean yield strength should be considered instead of the specified minimum yield strength for design. Mean yield strength of A36 steel is approximately 42 ksi, which was used to define the caisson material strength.

Figure 3.3 shows the structure computer model.



Figure 3.1—Example Caisson Platform Leaning Damage

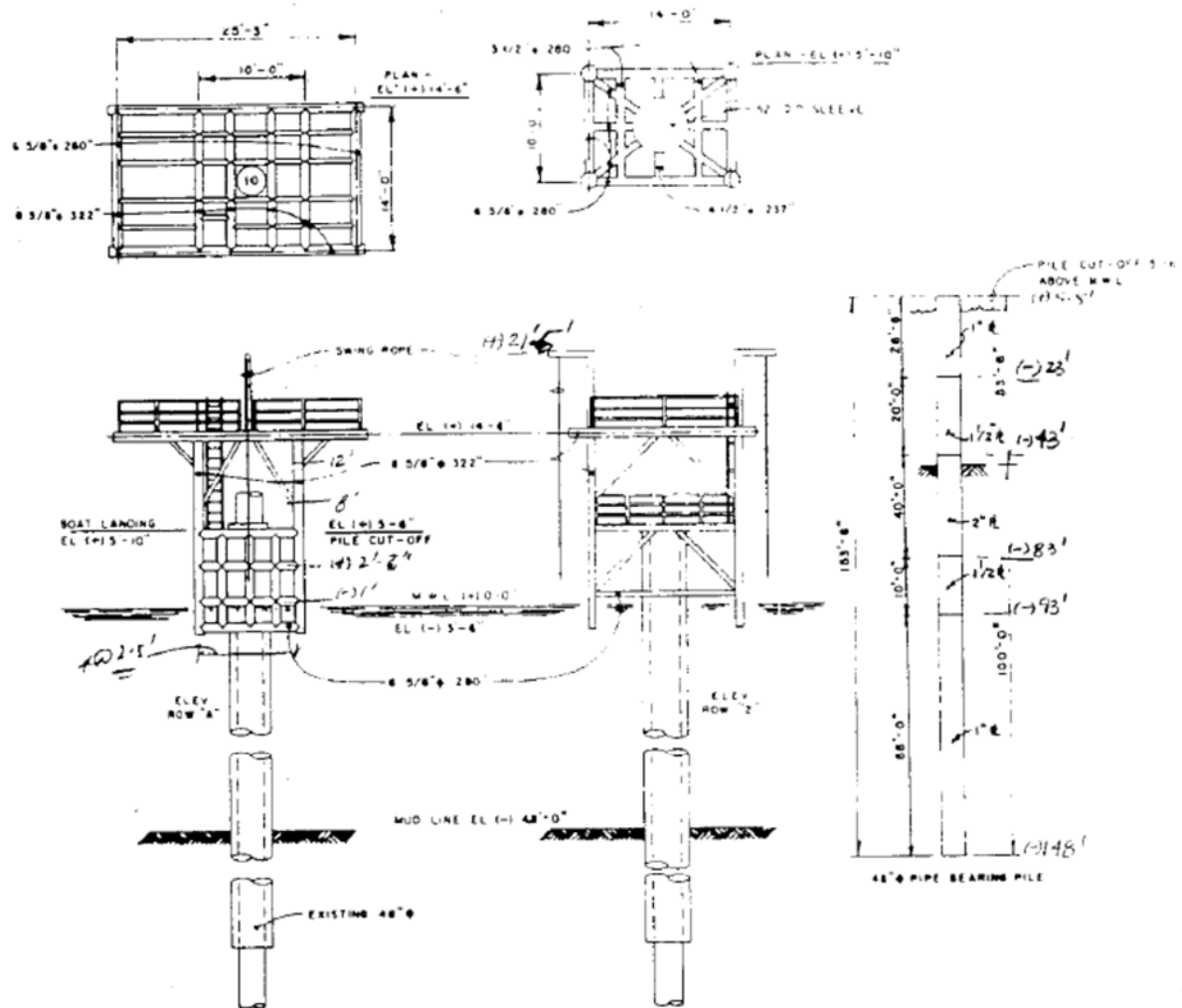


Figure 3.2—SS-Caisson Structural Detail

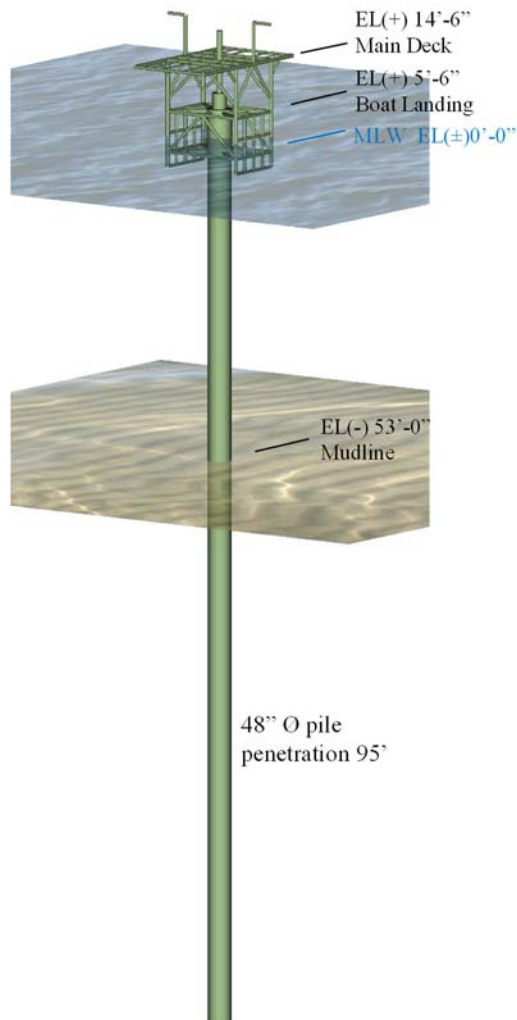


Figure 3.3 – SS-Caisson Structure Model

3.3 Site Soil Data

The original soil boring data collected in 1983 are site-specific. The caisson site mainly consists of soft clay in the top layers.

Historically, the shear strength of clay sample has been determined with different soil sampling and testing methods (Ref. 29). The updated API/ISO RP 2GEO, 2nd Edition formulations to calculate clay lateral resistance (p-y curves) require the determination of the undrained shear strength profile, based on the standard direct simple shear (DSS) testing on 3" diameter push-samples.

Therefore, the best-estimate DSS shear strength profile of the clays at site was determined from the original soil boring samples. Figure 3.4 summarizes the procedure and the proposed DSS strength profile. This interpreted DSS shear strength profile was used to calculate the clay p-y curves for both 2GEO, 1st Edition and 2nd Edition draft. Details of the soil boring data and shear strength calculations are documented in Annex A for reference.

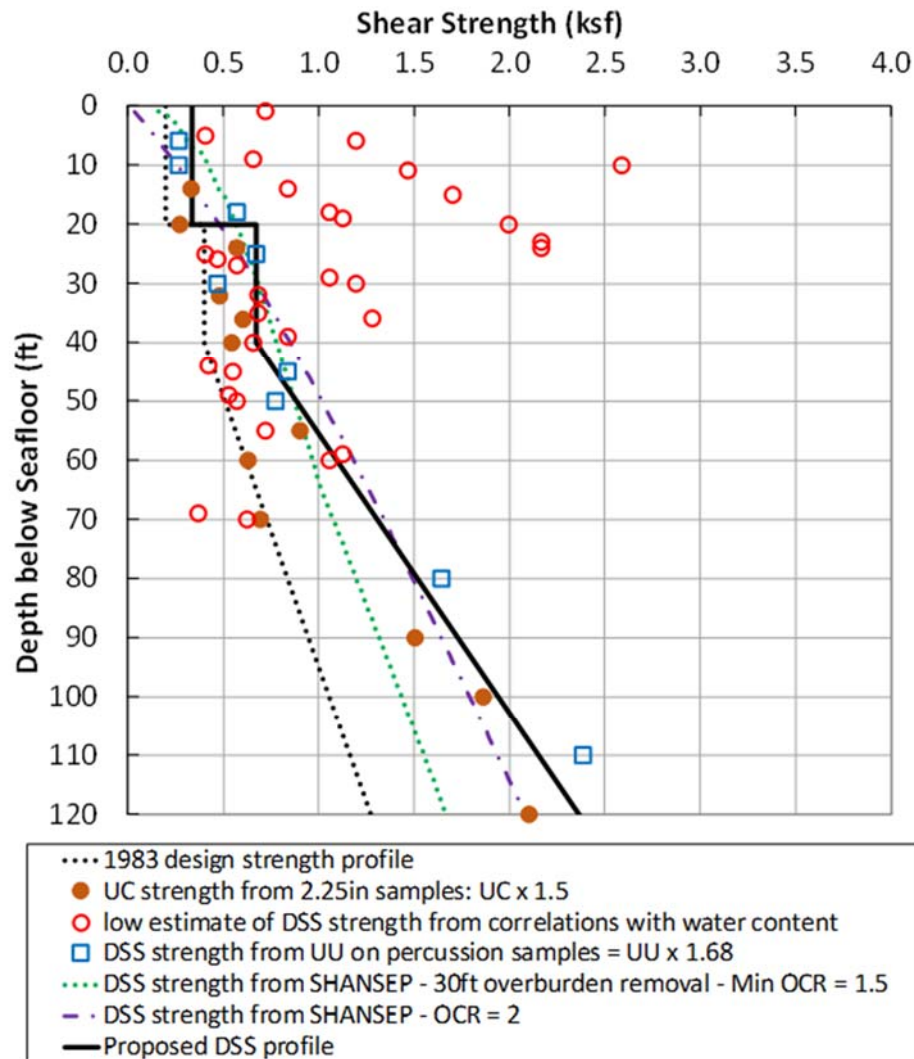


Figure 3.4—SS-Caisson Soil DSS Strength Profile

Two (2) sets of clay p-y curves were calculated and modeled at multiple depths of the soil layers:

- p-y curve model 1, static p-y curves based on current API/ISO standards, i.e. API RP 2A and 2GEO, 1st Edition.
- p-y curve model 2, cyclic p-y curves based on proposed API/ISO update, i.e. API RP 2GEO, 2nd Edition (Draft).

2GEO, 2nd Edition has provided guidance and comparison of p-y curves developed from both 2GEO, 1st and 2nd Editions. The difference of these two (2) sets of p-y curves developed for the site-specific soil are also compared in Annex A.

Other soil spring curves (t-z & Q-z curves) were also created according to the formulations of corresponding standards. These t-z and Q-z spring curves are generally identical for the two (2) soil models.

Therefore, two (2) structural models were developed, with identical structural modeling detail but using different soil spring models.

3.4 Hurricane Andrew Hindcast Data

Hurricane Andrew metocean data at the caisson site were primarily based on the GOMOS2014 hindcast data by Oceanweather. GOMOS2014 metocean data include tropical storm hurricanes from 1900 to 2014. The hurricane metocean data provide seastate time series with a 15 minutes resolution for a 7 km grid. The grid points closest to the caisson site were identified and interpolated to define the site-specific sea state. Hurricane hindcast data from other sources (Ref. 24, 25) were also compared and referred. Table 3.1 provides a summary of Hurricane Andrew hindcast metocean.

Table 3.1—Hurricane Andrew Hindcast Metocean

Metocean	Hurricane Andrew Data
Storm tide (ft)	1.6
Wave height Hmax (ft)	42.6
Wave period Tmax (s)	12.2
Wave direction (deg)	249
Current speed Vs (ft/s)	6.0
Current direction (deg)	232
Wind speed Ws 1hr@10m (ft/s)	143.0
Wind direction (deg)	204
Notes: 1. Direction is “to which” direction, clockwise from true North in degrees. 2. Max wave height is close to breaking wave height. 3. Wave kinematics factor 0.88. 4. API hydrodynamics definition applied.	

3.5 Dynamic Pushover Analysis

An explicit dynamic pushover approach was adopted to predict the platform response to repetitive wave loadings in this study. In this approach, the hurricane waves were dynamically run through the structure in the time domain, pounding the platform repeatedly to accumulate deformation and/or damage on the structure, more realistic to reflect the actual event.

For comparison purpose, both static and dynamic pushover analyses were performed in this case to gain insight on the difference between these two (2) analysis approaches. Traditional static pushover is to monotonically ramp up wave loading on structure until its collapse in a quasi-static way. Dynamic pushover is to run the wave through the structure dynamically multiple times for repeated cyclic loading effect. In this study, the focus was to simulate the hurricane wave dynamic effect explicitly and best estimate the structural response, so as to make meaningful comparison to the real platform behavior in storm. Therefore, dynamic pushover approach, with both wave hydrodynamics and structural dynamics explicitly included, suited this purpose better to produce more realistic results. This is especially true for this single free-standing caisson case, with predicted structural response to be a progressive accumulating process. For comparison, static pushover analysis was first performed and showed that the caisson would have completely toppled on seafloor because the global bending moment created by wave loading exceeded the caisson pile full plastic hinge capacity.

3.6 Results and Comparison

3.6.1 General

The caisson topsides elevation, as shown in Figure 3.3, was relatively low and therefore was fully inundated below the waves when the hurricane hit the platform. For all waves with wave heights over 20 ft, the wave crests should exceed the main deck level. For such case, the caisson response to wave action would be similar for a range of waves with wave heights over 20 ft.

The Hurricane Andrew wave height at the site was actually maxed out to the shallow water breaking wave limit. Due to this fact, a large number of wave cycles close to the maximum wave condition, and their repeated impacts on the structure, should be expected at the time.

Given these considerations, multiple waves near the defined hurricane wave height were expected. In the dynamic analysis, 20 cycles of hurricane wave (see Table 3.1) were run through the structure to check their accumulated pounding effects on the platform. This is a simplified analysis approach as compared to the more realistic random sea analysis with wave trains. Probabilistic calculation of Hurricane Andrew wave cycles (using the Forristall distribution) indicates that the assumed 20 hurricane wave cycles reasonably represent the number of effective waves and their effect on the structure in the storm.

3.6.2 Model with API 2GEO, 1st Edition p-y Curves

Due to the relatively softer clay and p-y springs, the caisson pile yielded and formed a clear plastic hinge at about 40 ft below the mudline (at a weaker section of the pile) in the first one or two wave cycles due to the wave pounding loads. With the continued wave impacts in following cycles, the plastic hinge angle kept increasing in each cycle and the caisson global deformation and leaning angle kept developing. After about 20 wave impacts, the caisson tilted to approximately 35 degrees and should have almost toppled after this point.

Figure 3.5 provides a sequence of snapshots to show the platform response to wave impacts in multiple cycles. The platform is predicted to lean more than 35 degrees after 20 wave cycles and may have possibly toppled after Hurricane Andrew.

3.6.3 Model with Proposed API 2GEO, 2nd Edition p-y Curves

The caisson pile yielded initially along a segment 30 ft below the mudline in the first one or two wave cycles under the wave pounding loads. With the continued wave impacts in following cycles, the plastic hinge gradually formed at around 30 ft below the mudline and caisson global rotation started to accumulate. At around 20 wave impacts, the plastic hinge rotation stabilized at around 16 degrees of global tilting due to the stronger soil p-y springs in top layers. It is estimated that a balance between wave loading and soil resistance is achieved near this point.

Figure 3.6 shows a sequence of snapshots of the platform response to wave impacts in multiple cycles for this case.

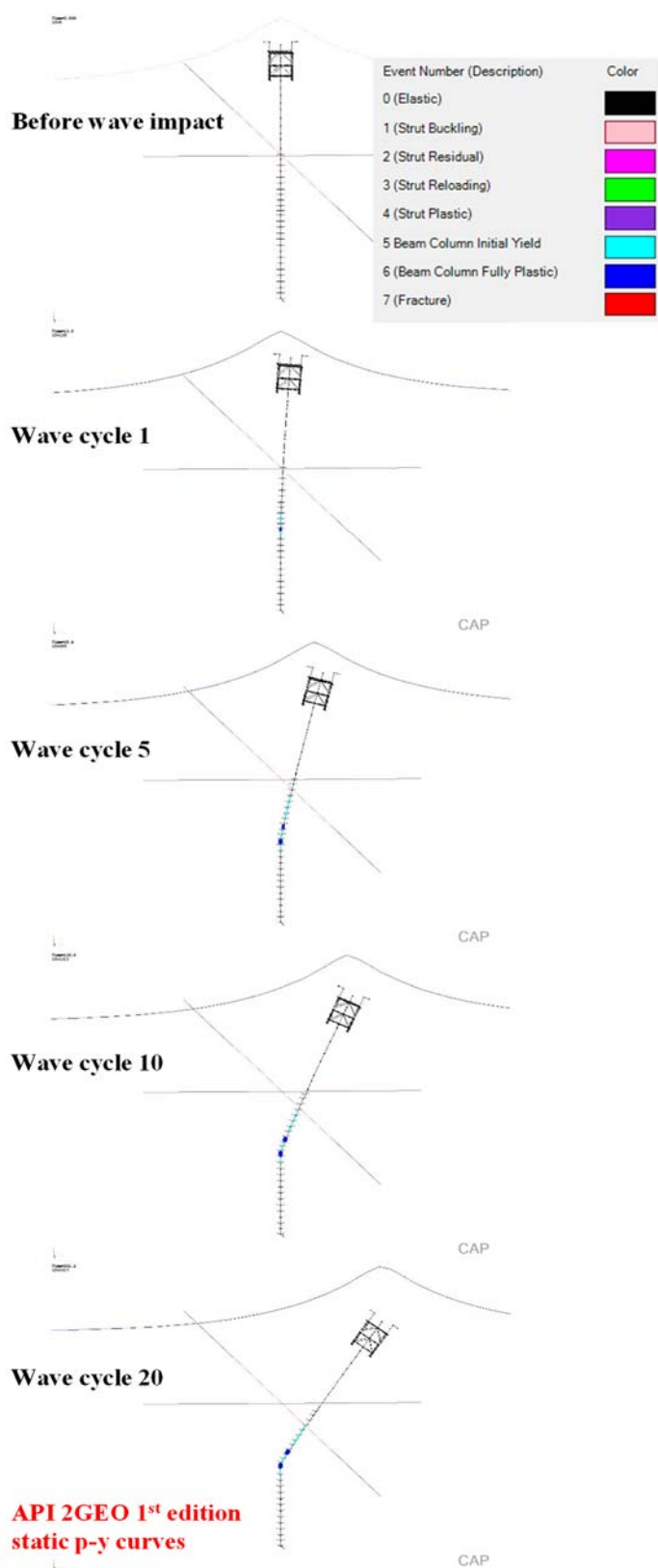


Figure 3.5—SS-Caisson Response to Hurricane Andrew Wave Impact, Model with API 2GEO, 1st Edition p-y Curves

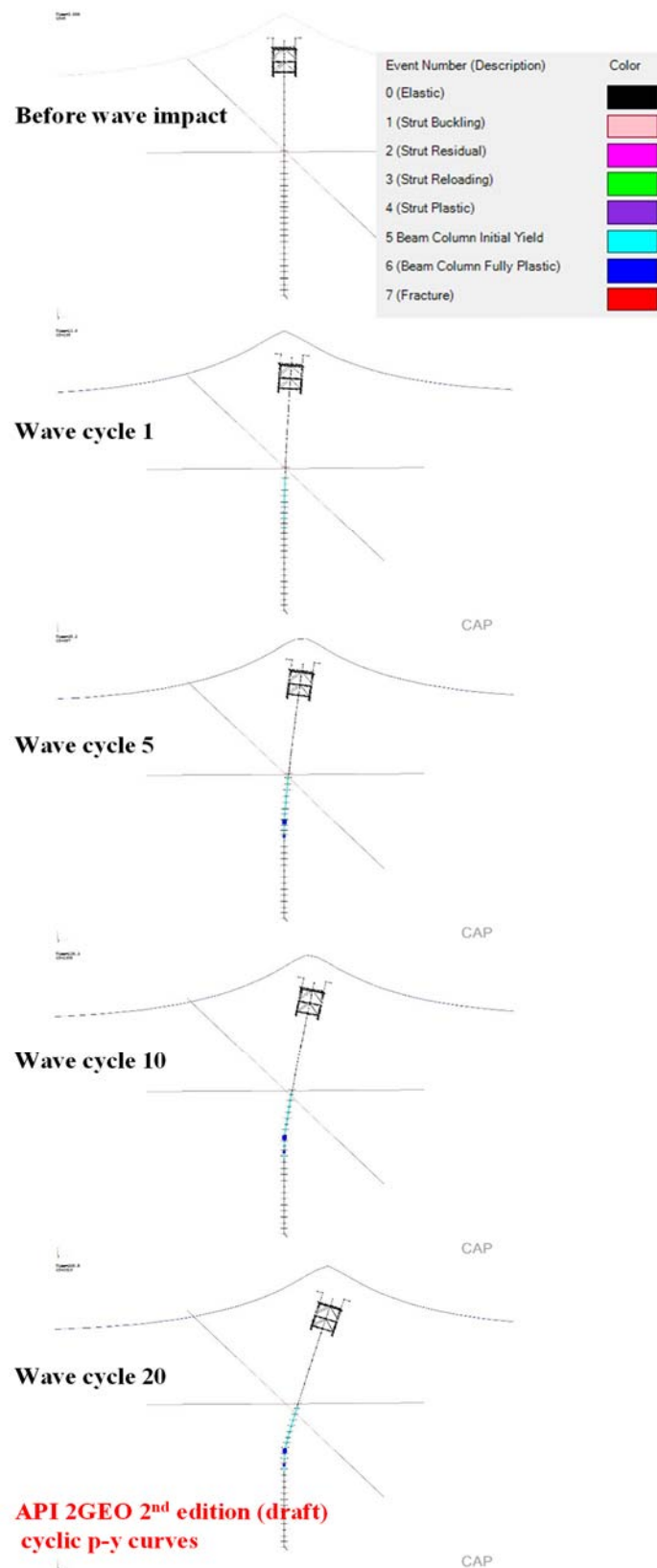


Figure 3.6—SS-Caisson Response to Hurricane Andrew Wave Impact, Model with Proposed API 2GEO, 2nd Edition (Draft) p-y Curves

3.6.4 Comparison

To compare the difference of two (2) soil models, the expected caisson behaviors and positions after Hurricane Andrew impact are presented in Figure 3.7 for these two (2) cases. Figure 3.8 provides the caisson hurricane wave load vs. global rotation curves of two dynamic pushover runs. The analysis clearly shows that the caisson performance predicted by the stronger 2GEO, 2nd Edition p-y curves closely matches the actual reported caisson leaning position after the hurricane. If the soil lateral resistance is close to the 2GEO, 1st Edition p-y curves, the caisson should have tilted much more or possibly toppled.

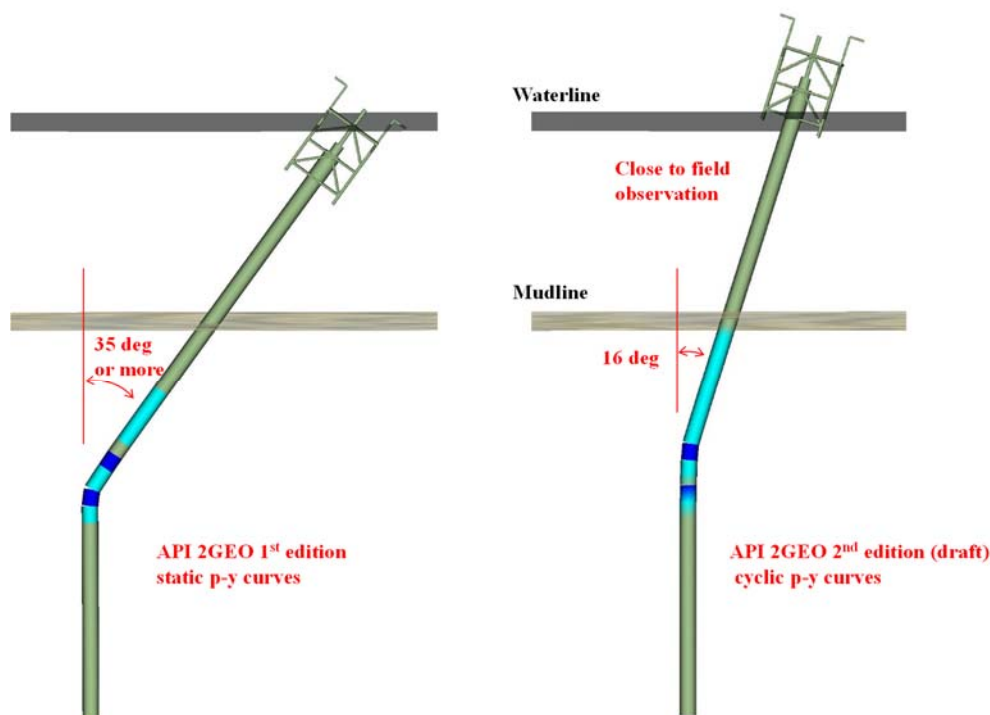


Figure 3.7—SS-Caisson Damage to Hurricane Andrew Wave Impact

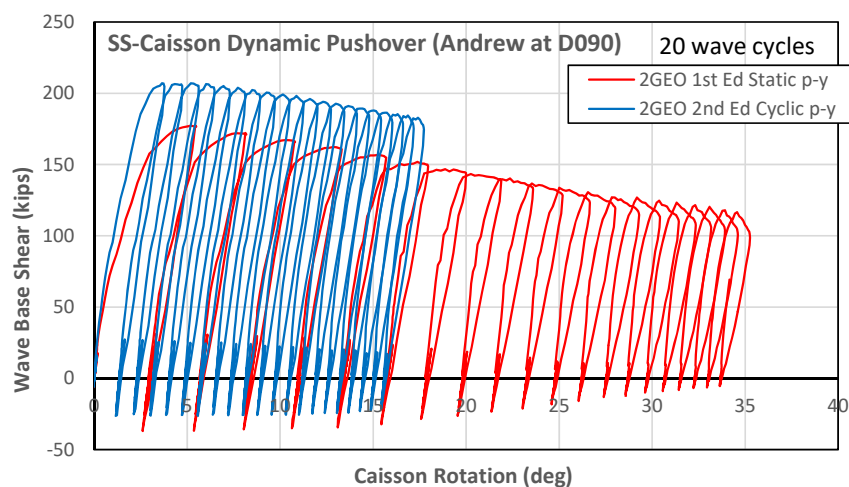


Figure 3.8—SS-Caisson Hurricane Wave Load and Rotation

4 Assessment of Jacket Platform Damage in Hurricane Ike

4.1 General

SS-Platform is a jacket type of platform located in West-Central region of the GOM. The platform was damaged during Hurricanes Gustav & Ike in 2008 (Ref. 12).

4.2 Jacket Platform Structure Detail

The SS-Platform is comprised of two (2) 4-pile jackets connected above waterline. The platform is located in a 100 ft water depth and the two (2) jackets were installed in 1969 and 1972.

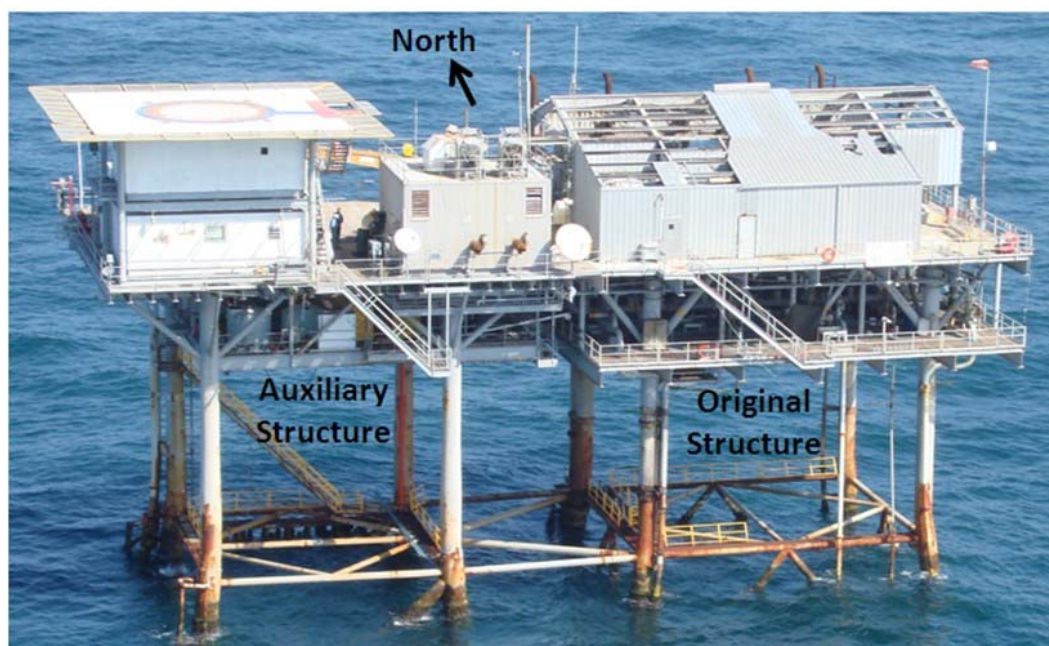
Figure 4.1 provides a picture of the platform after Hurricanes Gustav & Ike. Figure 4.2 shows the structural computer model and some of the structure features and details of the original design. The platform “as-was” condition before Hurricanes Gustav & Ike are considered. The major platform characteristics are:

- Combined two (2) 4-pile jacket platforms, jackets only connected above waterline at EL(+) 7’;
- Original jacket platform installed in 1969 on the East, auxiliary jacket platform installed in 1972 on the West;
- Water depth 100 ft;
- Original jacket piles (4) 36”Ø 0.5”~1.5”, auxiliary jacket piles (4) 42”Ø 0.75”~2.0”;
- Jacket vertical main bracing, K braces;
- Marine growth thickness based on historical underwater inspection measurements;
- Pile & jacket main frame material A36 steel;
- Pile soil penetration 180 ft, site soil—soft clay;
- Original platform cellar deck BOS EL(+) 40’, auxiliary platform cellar deck BOS EL(+) 43’ (both incurred wave-in-deck loads in Hurricane Ike);
- Topsides main deck at EL(+) 59’-6” above MLW level, total main deck size ~ 150’x 65’;
- Topsides total weight approximately 1500 kips (steel+equipment);
- Platform damaged during Hurricanes Gustav & Ike in September 2008, with tubular joint failures and member damage/buckling.

The original jacket has a vintage 1968 design. Its tubular joint and member design does not fully comply with the requirements of modern jacket design standards. This leads to untypical weak tubular joint and bracing system design, and also slender bracing members, as shown in Figure 4.2.

The platform was damaged during Hurricanes Gustav & Ike in September 2008. The damage was mainly concentrated on the original jacket structure, consisted of joint failures and member damage/buckling, as shown and described in Figure 4.3. The auxiliary jacket, though designed only a few years later, has an improved design more compliant to modern design concepts. Therefore, no significant structural damage was observed on that side of the platform.

For structural ultimate capacity assessment, the mean yield strength should be considered instead of the specified minimum yield strength for design. Mean yield strength of A36 steel is approximately 42 ksi, which was used to define the steel material.



SS-Platform Post Hurricane Ike

Figure 4.1—SS-Platform (After Hurricanes)

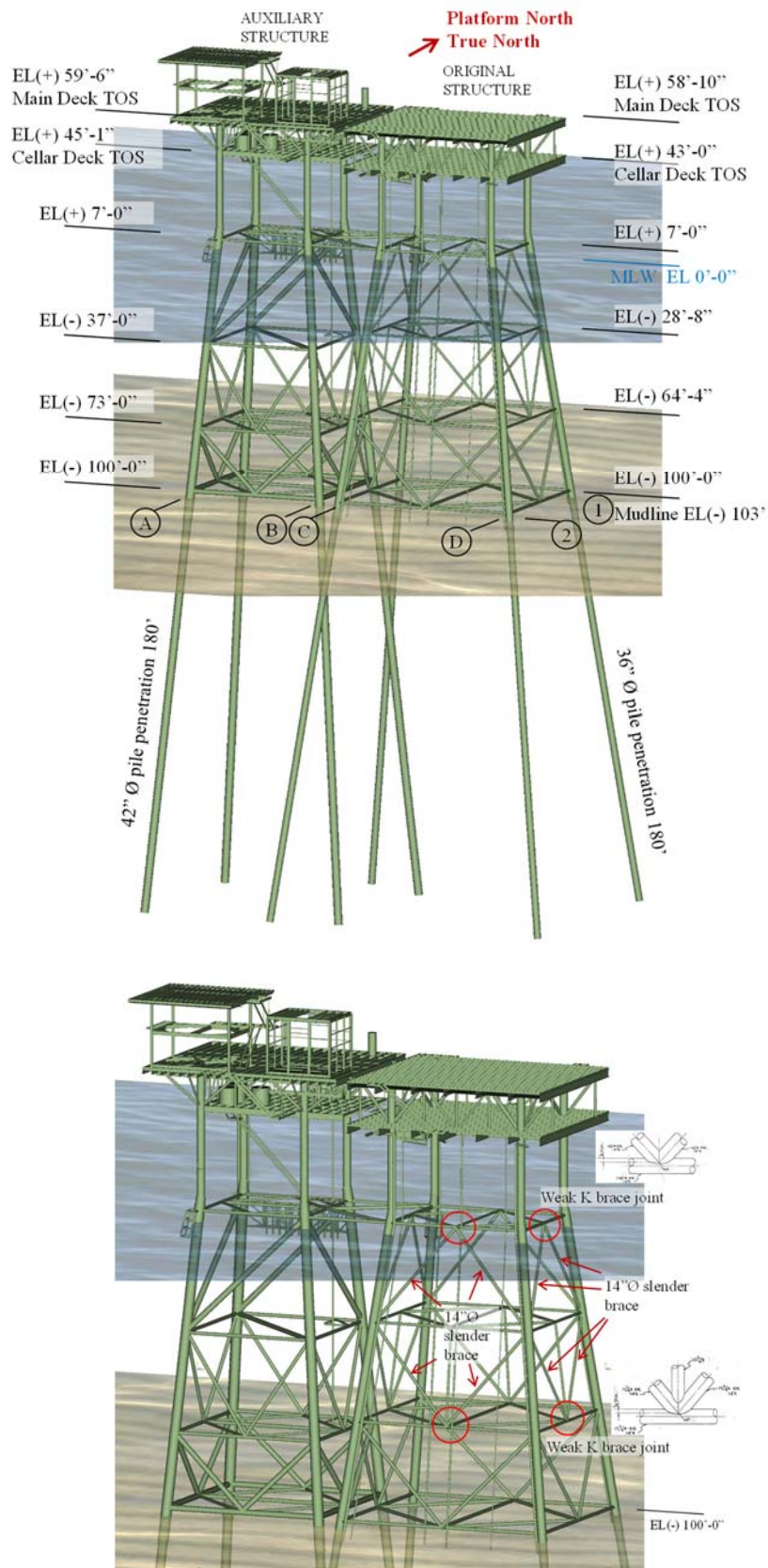


Figure 4.2—SS-Platform Structure Model and Design Detail

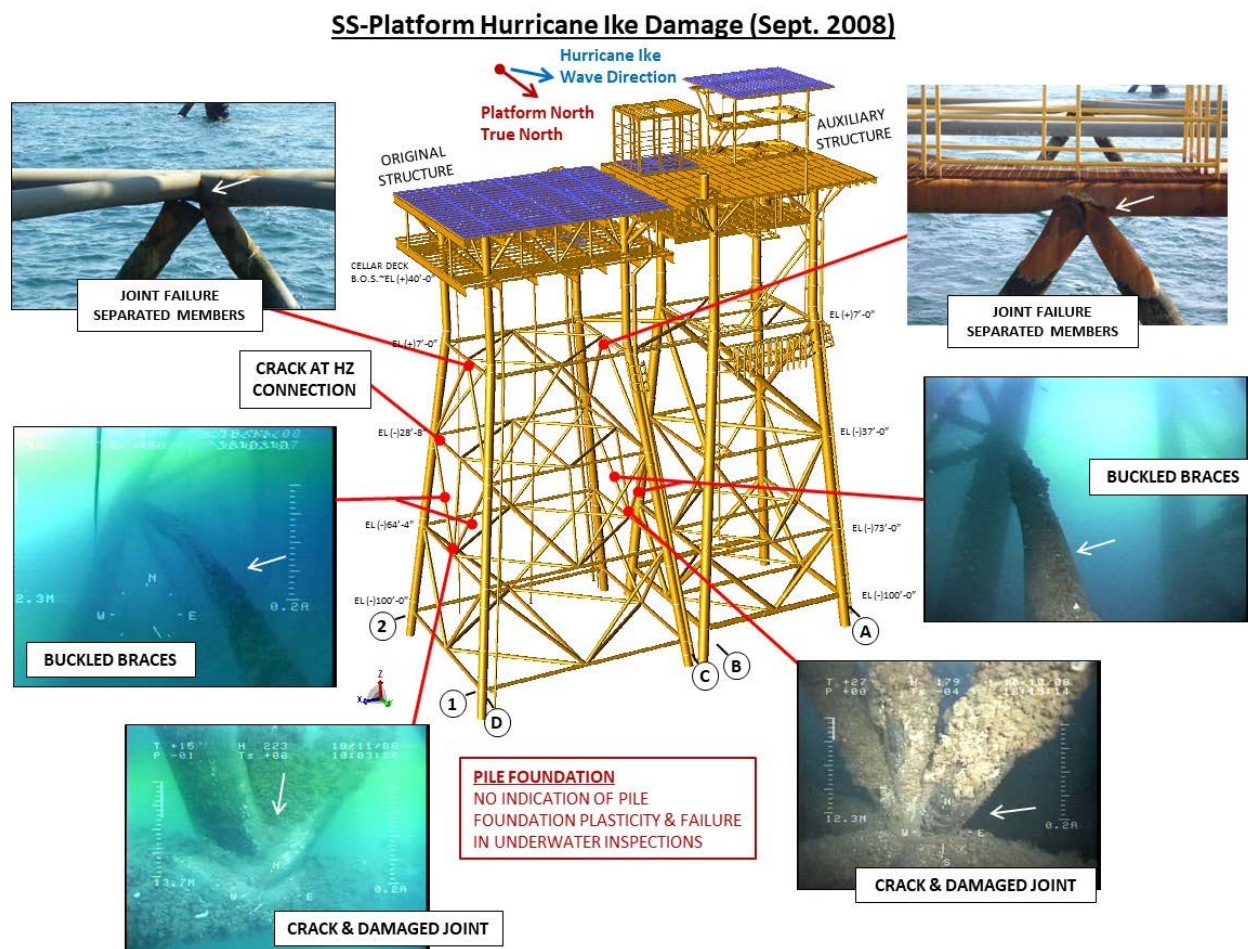


Figure 4.3—SS-Platform Damage After Hurricanes

4.3 Site Soil Data

The platform site soil boring data, dated in 1979, are based on a borehole percussion sampling at approximately 2,000 ft away from the site in the same GOM block. Three (3) other boreholes (push samples) within 3 miles are also referred and compared. All boring data are consistent and show similar clay profile and properties in top layers.

The updated API/ISO RP 2GEO, 2nd Edition formulations to calculate clay lateral resistance (p-y curves) requires the determination of the undrained shear strength profile, based on the standard direct simple shear (DSS) testing on 3" diameter push samples.

Similar to the caisson case, the best-estimate DSS shear strength profile of the clays at site was determined from the original soil boring samples. Figure 4.4 summarizes the procedure and the proposed DSS strength profile used to calculate soil clay p-y cures (for both 2GEO, 1st Edition and 2nd Edition draft). Details of the soil boring data and shear strength calculations are documented in Annex B for reference.

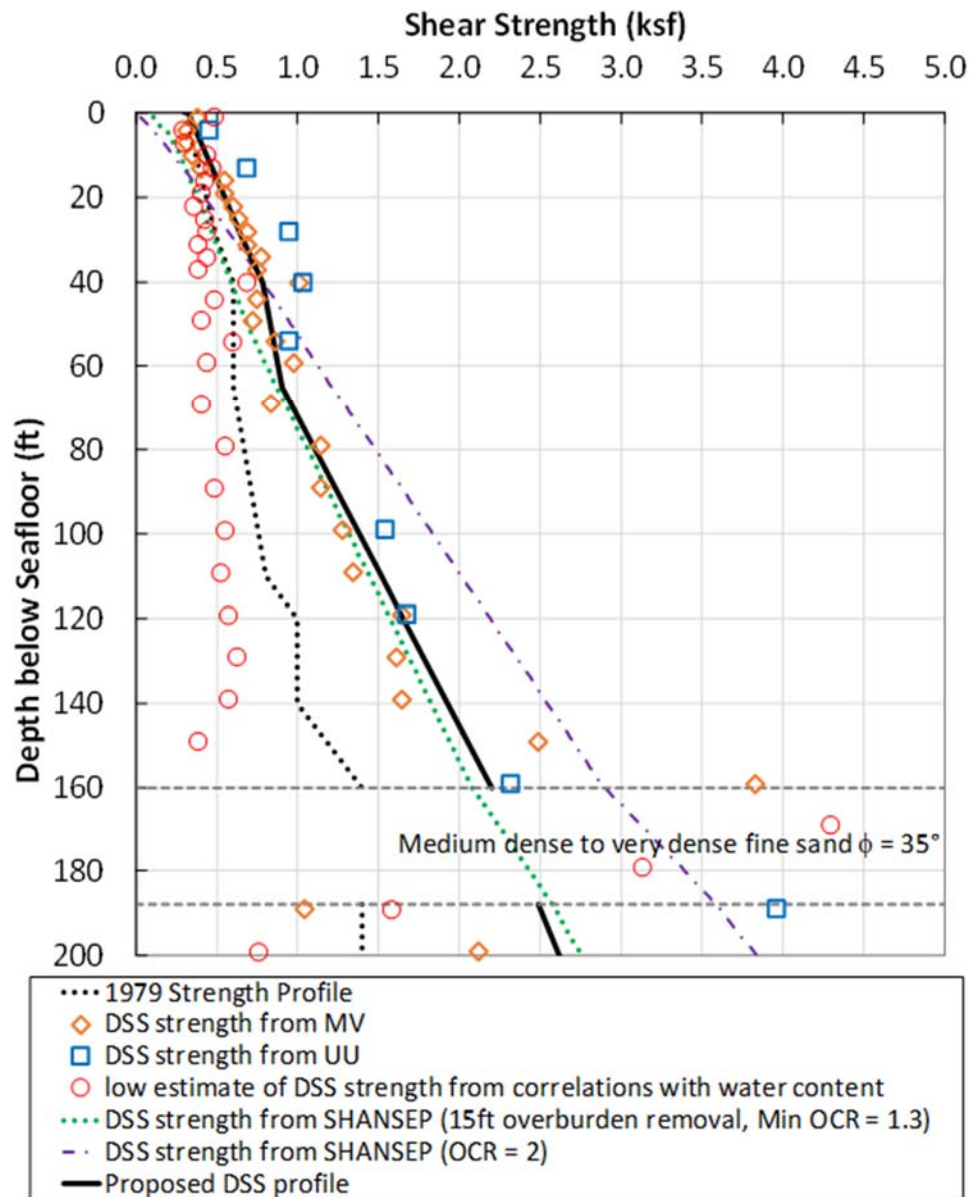


Figure 4.4—SS-Platform Soil DSS Strength Profile

Two (2) sets of clay p-y curves were calculated and modeled at multiple depths of the soil layers:

- p-y curve model 1, static p-y curves based on current API/ISO standards, i.e. API RP 2A and 2GEO, 1st Edition;
- p-y curve model 2, cyclic p-y curves based on proposed API/ISO update, i.e. API RP 2GEO, 2nd Edition (Draft).

Other soil spring curves (t-z & Q-z curves) were also created according to the formulations of corresponding standards. These t-z and Q-z spring curves are generally identical for the two (2) soil models.

Therefore, two (2) structural models were developed, with identical structural modeling detail but using different soil spring models.