16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 3524
Registration Code: S-I1463168687

NONLINEAR TIME HISTORY ANALYSIS FOR ASSESSMENT OF AN EXISTING OFFSHORE JACKET PLATFORM

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Abstract

It is often necessary to reassess the seismic performance of existing platforms due to changes to platform deck loading, increase in number of well conductors, or because modern characterizations of earthquakes result in more severe ground motions than were considered at the time of the design of existing platforms (20-30 years ago).

The primary goal of seismic reassessment of an existing platform is to assure that the platform system will have sufficient strength and ductility to withstand the Ductility Level Earthquake (DLE) without collapsing, although structural damage may occur.

For this study an existing jacket platform off the coast of California was selected. The seismic analyses were based on new design response spectra and three sets of conforming ground motions for the 1000-year DEL. They were developed using the New Generation of Attenuation Relationships considering the influence of near-source and rupture directivity effects. Ground motions were developed for non-liquefied and liquefied conditions.

The seismic assessment was carried out in three phases. The first phase of the study focused on the assessment of the foundation of the platform for the new DLE ground motions. Sensitivity analyses were undertaken to determine the effect on the platform's response to variations in key parameters such as liquefaction and, structural and soil damping. The second phase assessed the platform with an additional 18 conductors. In the final phase of the study, analyses were conducted to determine the Reserve Strength Ratio (RSR) by assessing the sensitivity of the structure and its foundation to the intensity of the motions and to soil capacity. The results of the analyses are presented in this paper.

Keywords: offshore platform, seismic evaluation, nonlinear time history, analysis, reserve capacity

1. Introduction

Given that modern characterizations of earthquakes result in more severe ground motions than were considered in the design of platforms 20-30 years ago, a seismic reassessment of these platforms is often warranted [2, 3, 4, 5, 6]. Present seismic design practice for offshore platforms considers two hazard levels; Strength Level Earthquake (SLE), and Ductility Level Earthquake (DLE). Reassessment methodology however, concentrates on failure prevention under DLE conditions. Therefore, the objectives of the seismic reassessment are to confirm the platform stability under DLE conditions, and to investigate the failure modes.

There are different techniques to calculate the seismic demand on structures such as response spectrum analysis and nonlinear time-domain analysis. Response spectrum analysis is a widely used procedure for demand calculation; however, it assumes that the structure possesses linear stiffness and damping. Yielding and stiffness degradation within a mode is impossible to consider explicitly, as it is not known when yielding will occur and how the modes will be affected. For systems possessing variable mass (such as the changing mass of fluid which interacts with a submerged structure), frequency-dependent damping (such as the damping properties of foundations) and non-linear stiffness (the non-linear force-displacement relationships of either a foundation element or a yielding structural element), approximations with limited accuracy must be introduced in order to capture these effects within the framework of modal analysis. Therefore for a system with numerous nonlinearities, nonlinear time history analysis is a more appropriate method to use. Nonlinear time-history analysis gives estimates of member forces and mass displacements for each discrete time step; as the equations of motion governing the system's masses are being solved directly at each time step, the stiffness and damping properties of the structure may possess significant non-linear force-displacement relationships. Examples would be elements which allow for changes in strength and stiffness over time.

In offshore platforms, nonlinear response of piles is the most important source of potentially nonlinear behavior of the system due to earthquake excitations. It is often necessary to perform nonlinear time history analysis of offshore platforms to account for soil nonlinearity, energy dissipation through soil radiation damping and structural nonlinear behavior of the piles.

An existing jacket-type offshore platform off the coast of California in more than 200 feet of water is considered for seismic reassessment to determine if the platform can withstand the Ductility Level Earthquake without collapsing. The primary focus of this work was to determine the level of performance of the existing platform's foundation, including the soil capacity and the piles' structural adequacy during rare, intense seismic events. During this study a sequence of analytical development steps was adopted prior to the dynamic time-history analysis of the platform-foundation system. Dynamic characteristics of the platform were determined using the eigensolution technique and these results were used to perform a frequency-domain analysis of the platform to develop pseudo-static loads for pushover analyses of the platform.

2. Geotechnical Data

Probabilistic seismic hazard analyses (PSHA) were performed to estimate the ground motions and to develop design response spectra for the 1,000-year Ductility Level Event (DLE) in accordance with API RP2A [1,7], using the New Generation of Attenuation Relationships and considering the influence of near-source and rupture directivity effects (see Fig. 1). Three sets of pile-head ground motions (total of six input motions) compatible with the DLE design spectrum were developed (see Table 1). Pile-head, or mudline, acceleration time histories for the HE140 ground motion for liquefied and non-liquefied conditions are presented on Fig. 2. For modeling the soil-structure-interaction depth varying axial (skin friction, or t-z, and end-bearing, or Q-z) and lateral (p-y) soil springs for the best-estimate soil strength profile and liquefiable soil were provided by the Geotechnical Engineer [8].

Table 1- Summary of Selected Motions Characteristics

Set	Earthquake	Magnitude	Distance (km)	Designation
1	Loma Prieta, USA	6.9	12.7	G00, G90
2	1979 Imperial Valley, USA	6.5	9.3	HE140, HE230
3	Loma Prieta, USA	6.9	13.0	STG00, STG90

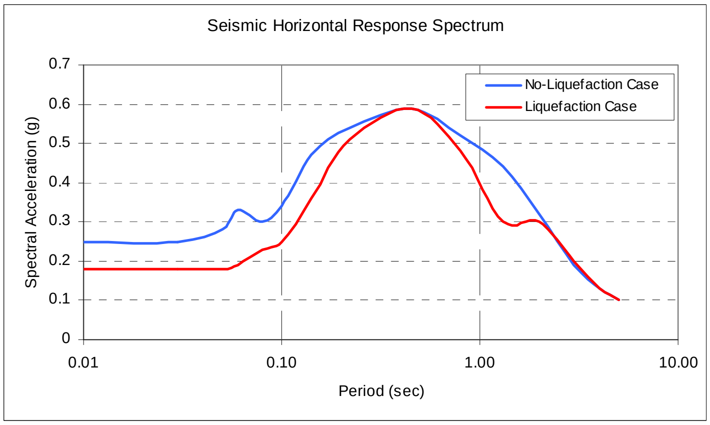


Fig. 1- Acceleration Spectra at Mudline for Non-Liquefied and Liquefied Conditions

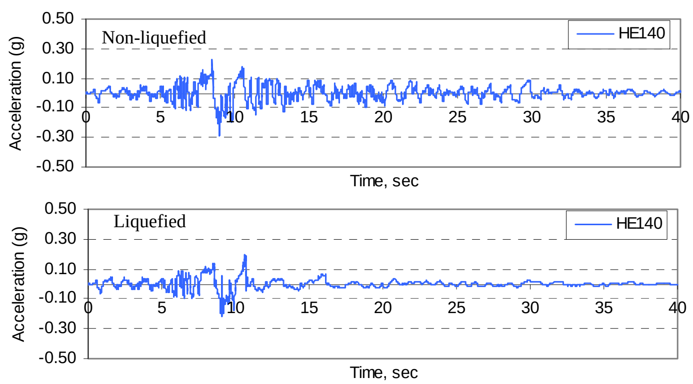


Fig. 2- Acceleration Time Histories at Mudline - HE140 Motion

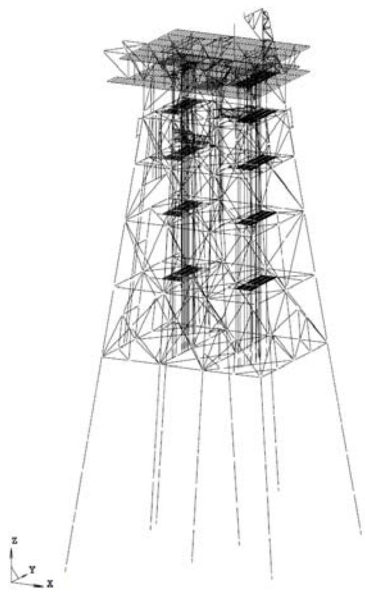
3. Modeling

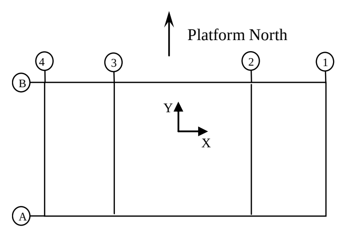
The considered platform is a steel, tubular-membered, truss-framed structure supported on eight tubular batter piles with pile penetrations varying from 170 ft to 230 ft. The piles are 60-inch in diameter with wall thickness varying from 1.5 inches to 2.5 inches. The piles are extended into the jacket legs and are cut-off 3 feet above the jacket legs. Since the focus of this study was to determine the performance of the platforms' foundation, the following were not included in this assessment:

- Secondary structural elements, including plan conductor framing, riser supports, boat landing and deck appurtenances
- Deck beams (truss girders were included in our assessment)
- Appurtenances and their connections to the platform
- Detailed modeling of the well casings inside the conductors
- Capacity of the pile connections to the platform
- Impact of coincidence of wind and/or wave loads in combinations with the seismic event
- Check for the operating basis earthquake
- Pile corrosion.

The platform and deck loadings were modeled in computer program SAP 2000 [9] for the initial assessment of the platform for earthquake, with a simplified deck structural model. The secondary structural members in the decks were not included. However, the model with modified deck has the same stiffness and mass as the full deck model by adding braces at deck levels and lumped mass at leg nodes, respectively.

For comprehensive verification purposes, a second program, Extended Design Program (EDP) was used to independently analyze the platform. During the initial assessment of the platform both programs were used in parallel to increase the confidence level in the results. Thereafter, all the analyses were carried out using EDP computer program. EDP [10] was developed in the early 1980’s particularly for the analysis of offshore platforms in highly seismic zones. The EDP model included piles, jacket, and deck primary and secondary framing including joint cans as well as platform appurtenances including two boat landings, barge bumpers, risers, sumps, vent boom, pump casings, sewer casings, etc. (see Fig. 3).





Platform Orientation

Fig. 3- Isometric View of the Platform, EDP Model

3.1 Foundation modeling

3.1.1. Linear model

Linearized representations of the foundation are required for the linear eigensolution and response spectrum analyses. The program Soil-Pile-Interaction (SPI), Version 3.3 was used to develop linearized foundation springs at mudline. The eight-pile foundation was grouped into two sets: corner piles and inner piles. Springs were developed in SPI for each set, for both non-liquefied and liquefied conditions.

3.1.2. Nonlinear model

In non-linear analysis, the soil-structure interaction was modeled by sets of non-linear soil springs connected to the element nodes of discretized piles, which model the soil/pile response.

The lateral soil (p-y) strength was modeled by a series of multi-linear hysteretic springs. The curve presented in Fig. 4 defines the force-deformation relationship under uniaxial loading. The first slope on either side of the origin is elastic; the remaining segments define plastic deformation. Soil skin friction was also modeled by a series of multi-linear hysteretic springs representing the t-z soil springs, as shown in Fig. 4. The end bearing soil springs were modeled as “Multi-Linear with Elasticity Property”. The behavior is nonlinear but it is elastic. This means that the element loads and unloads along the same curve, and no energy is dissipated.

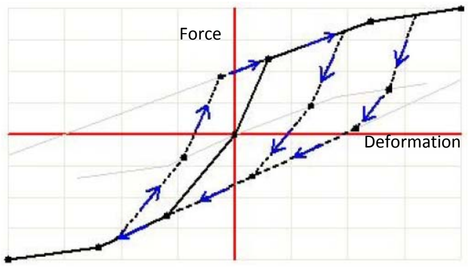


Fig. 4- Multi-Linear Kinematic Plasticity Property Type for Uniaxial Deformation (Source: SAP2000 Manual)

4. Analysis Procedure and Results

4.1. Eigensolution

The dynamic characteristics of the platform were determined using the eigensolution technique and these results were used to perform a frequency-domain analysis of the platform to develop pseudo-static loads for pushover analyses of the platform. Eigenvector analysis was performed to determine the undamped, free-vibration mode shapes and natural frequencies of the platform, which were used as the basis for response spectrum analyses. The natural periods and mass participations of the primary modes of the platform with the linearized foundation stiffness for non-liquefied and liquefied conditions are presented Table 2.

Table 2- Natural Period, Linearized Foundation

Soil Condition	Period		
	X	Y	Z
Non-Liquefied	2.22 sec	2.10 sec	0.53 sec
Liquefied	2.51 sec	2.43 sec	0.54 sec

4.2. Response spectrum

The response spectrum analysis was performed for 24 directions (every 15 degrees) to determine the critical direction of the structural response for the piles. The linearized foundation stiffness matrices were applied at mudline. The results indicate that the maximum pile loading occurs when the loading is applied at 330 degrees counterclockwise from platform North. A summary of the response spectrum analysis results is presented in Table 3. In comparison with the non-liquefied case, a 10% decrease in resultant base shear and over 15% decrease in overturning moment is observed under liquefied conditions.

Table 3. Base Shear and Overturning Moment at Mudline, 330 degrees loading direction

Case	Base Shear (kip)			Overturning Moment(kip-ft)		
	F_X	F_Y	Resultant	M_X	M_Y	Resultant
Non-Liquefied	9,145	5,624	10,735	1,259,122	-2,098,601	2,447,348
Liquefied	8,329	4,967	9,698	1,019,583	1,794,122	2,063,595

4.3 Pushover analyses

The capacity of the structure-foundation system to withstand horizontal loads and displacements is determined by pushover analysis. The pile foundation is modeled by sets of beam elements connected to non-linear p-y, t-z, and q-z springs along the piles, for non-liquefied and liquefied conditions. A set of horizontal loads, which simulate the inertial loads that act during an earthquake, was applied and incrementally increased. The applied forces were based on the response spectrum analysis. These forces were applied at the leg nodes and incrementally increased.

4.3.1. Non-Liquefied

The structure's ultimate displacement capacity is about 44 inches (this is a relative displacement between deck and pile tips). The skin friction capacity of three corner piles is reached at 22 inches of deck horizontal displacement; however, the inner piles have reserve skin friction capacity. The first pile hinges form in piles B1, B2, and B4 at about 46 ft below mudline at a deck displacement of 44 inches.

4.3.2. Liquefied:

The structure's ultimate displacement capacity is about 35 inches. The skin friction capacity of three corner piles is reached at 23 inches of deck horizontal displacement, however, the inner piles have reserve skin friction capacity. The first hinges form in piles B1 and B4, at about 40 to 50 ft below mudline at a deck displacement of 23 inches. It is observed that hinging in piles occurs earlier for liquefied conditions than for non-liquefied conditions. The resultant base shear versus displacement response of the platform at deck level for liquefied condition are shown on Fig. 5 for the critical diagonal direction, which causes most load on the corner piles.

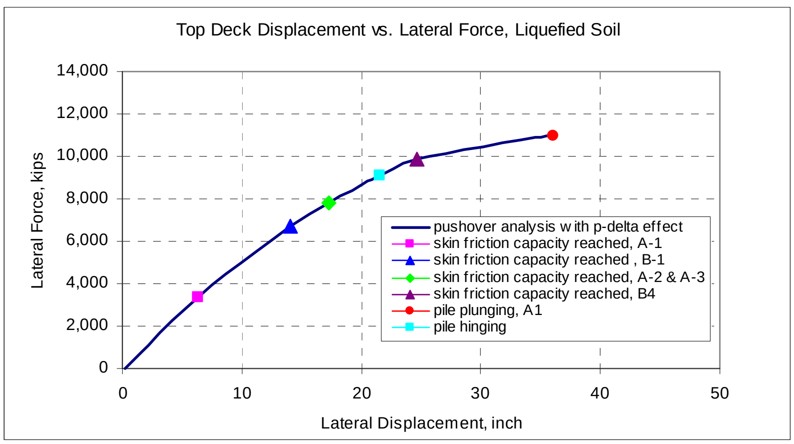


Fig. 5- Pushover Analysis Results, Base Shear vs. Deck Displacement, Liquefied Case

4.4. Time history analyses

Nonlinear time-history analyses for all three sets of motions were completed, applying the fault normal and fault parallel components in each of the two main axis orientations along the length of the piles as time- and depth-varying displacements of the soils. The time-history analyses were run with and without vertical motions, in order to understand the importance of vertical motions. Thus, a total of 4 analyses were completed for each of the three ground motions, or a total of 12 time-history analyses. The motions were applied to the piles through a series of non-linear hysteretic springs that represent soil friction, lateral stiffness and pile tip end-bearing response. For the liquefied condition, the analysis was carried out with the reduced soil springs and liquefied motions.

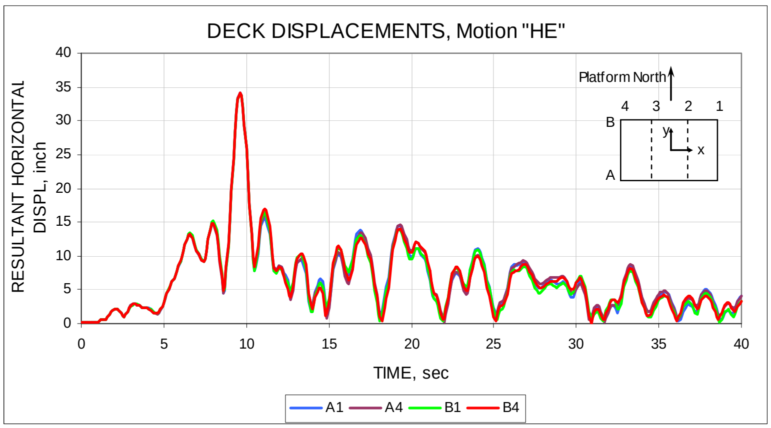
The nonlinear time-history analyses were performed in three phases. The first phase of the study focused on the assessment of the foundation of the existing platform, but for the new DLE ground motions. Several sensitivity analyses were undertaken during this phase. This was followed by a second phase, which dealt with the assessment of the platform with the future additional conductors. In the final phase of the study, analyses were conducted to determine the Reserve Strength Ratio (RSR) by determining the sensitivity of the structure and its foundation to the intensity of the motions and to the soil strength. The results of the time-history analyses are presented below.

4.4.1. Phase I: Non-linear Time-History analysis of the Existing Platform

The primary focus of this work was to determine the level of performance of the existing platform's foundation, including the soil capacity and the piles' structural adequacy. The results of these analyses are presented below.

Displacements: The maximum resultant horizontal transient motion of the main deck from its initial position during any of the earthquakes is about 34 inches absolute and the base of the platform experiences resultant horizontal motion of about 21 inches. The maximum transient absolute vertical movement of the platform, measured at its main deck, is about 8 inches. The horizontal permanent set of the deck, after the platform comes to rest, is about 4 inches. The deck elevation is about 4 inches lower after the worst of the earthquakes than before. Time-history plots of the resultant horizontal absolute displacements at deck level for corner legs are presented on Fig.6. The platform displaced shape at the peak transient and at the end of shaking are shown in Fig. 6.

Pile structural behavior: The analyses indicate that local yielding of piles occurs during the maximum excursion cycle. Piles A1 and B1 are affected 40’ to 50’ below mudline, while piles B2 and A3 yield at a location 30’ to 40’ below mudline. Pile B4 also exhibits some yielding at a lower depth. Full plastic hinging does not form at any of these locations. This condition does not represent structural failure of the piles, but it does indicate that the limits of the platform foundation are being approached. For this reason further analysis was recommended, with all proposed additional conductors added and a check of the platform’s sensitivity to the predicted level of shaking.



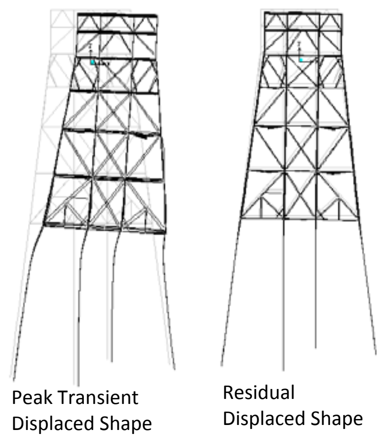


Fig. 6- Time History of the Absolute Resultant Horizontal Displacements at Deck Corner Leg Nodes, Motion HE , No Liquefaction

Soil demands: The largest compression force (8,525 kips) in a pile occurs in pile A1 during the HE earthquake motion. This load exceeds the pile’s friction capacity and the pile tip subsequently moves downward to a displacement, relative to the soil, of 2.6 inches. The pile then returns upward and at the end of the earthquake, it remains about 2 inches below its initial elevation. The maximum tension force in any pile occurs in pile B4 and is about 6,080 kips, which approaches its tensile capacity and slightly exceeds its skin friction capacity. The corner piles dominate the behavior of the platform foundation rather than the inner piles, which serve mainly to support the weight of the platform and to resist its horizontal movement. The corner piles’ capacity to resist downward and upward loads is most important to the prevention of overturning of the platform. It is important that when the platform vibration reaches its extreme motions, the skin friction capacity of some of the piles is exceeded for periods up to 1.0 second. This does not mean that the total foundation system reaches its capacity, because the 8-leg pile system has redundancy beyond that of a single pile. Also, while much of the energy of the vibration is associated with overturning of the platform, a significant amount of the energy is associated with horizontal sway vibration of the platform which causes bending in the piles, not axial loading.

A time-history plot of the corner and inner pile axial force demand /capacity ratio is provided in Fig.7 and Fig.8, respectively. The compression capacity is the sum of skin friction along the pile and end bearing at the pile tip. The tensile capacity includes soil friction capacity and a reverse end bearing component (suction capacity), which is approximately equal to 15% of the end bearing capacity in compression. These are both represented by the 1.0 on the vertical axis in Fig.7 and Fig.8. The dotted lines on those figures indicate when the skin friction only of the pile would be exceeded. The total capacity is comprised of about 55% skin friction and 45% end bearing. The skin friction is the first component to reach its capacity, then the end bearing approaches its capacity at a later loading.

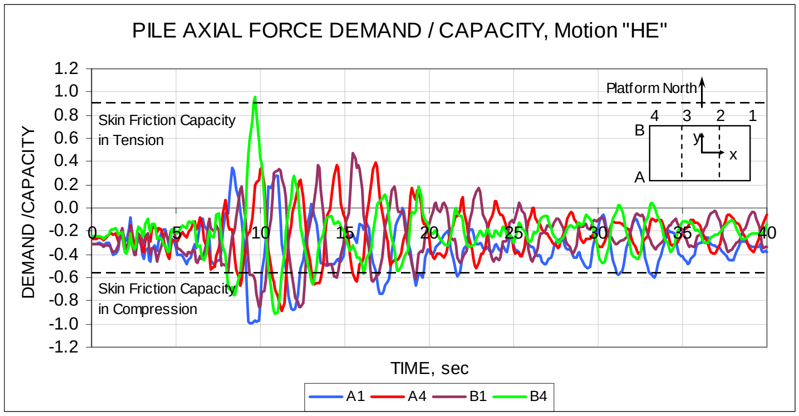


Fig. 7- Time-History of the Pile Axial Force D/C Ratio for 4 Corner Piles, Motion HE, No Liquefaction

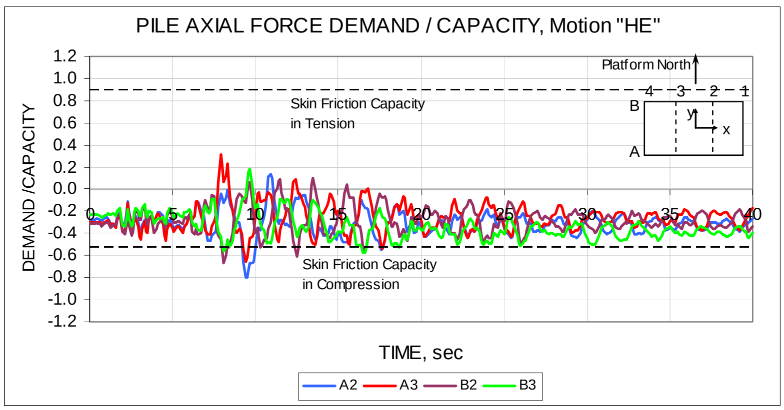


Fig. 8- Time-History of the Pile Axial Force D/C Ratio for 4 Inner Piles, Motion HE, No Liquefaction

Several sensitivity analyses were undertaken to determine the effect on the platform's response to variations in key parameters. These included the effects of (1) liquefaction; (2) structural damping, where the HE motion was re-analyzed with reduced structural damping of 3%, as compared to the 5% used throughout this study; (3) a blanket +25% and -25% variation in soil strength, and (4) no hysteresis considered in the t-z and p-y springs.

Effect of liquefaction: The HE motion was re-analyzed using the soil springs for the liquefied condition. The maximum displacements are slightly lower for the liquefied case, with the exception of the peak transient horizontal displacement at mudline. Force comparisons indicate that the maximum pile head axial forces are lower for the liquefied case.

Effect of Structural Damping: A reduced structural damping of 3% led to larger displacements, but only marginally. Typically, the largest percent increase occurred for residual horizontal displacements, but these are for fairly small absolute values (1 to 4 inches of resultant displacement). A 1% to 4% increase in forces is noted for the reduced damping case.

Effect of Variability in Soil Strength: With 25% softer soils, the deck horizontal displacement does not change significantly from the base case (it reduces). The maximum deck residual vertical displacement increases by about 1 inch. There is significantly less yielding in the piles for the reduced soil strength case. When 25% stronger soils are assumed, the deck horizontal displacement does not change. The maximum deck residual vertical displacement decreases by less than 1 inch. Yielding approaching a full plastic hinge is expected in pile A1.

Effect of Assuming No Hysteretic Damping in the Soil Springs: This sensitivity case was run to confirm that the predicted stability of the platform following the DLE motions is not a direct result of the assumptions made in the hysteretic damping behavior of the foundation. All hysteresis effects were turned off, such that the p-y, t-z, and Q-z soil springs were defined as non-linear elastic springs. The results of this analyses were markedly similar to the results of the analyses that assumed hysteretic damping, indicating that the large hysteretic loops that occur during the larger pulses are not the main reason for the structure recovering and returning to near its initial position after the earthquake.

4.4.2. Phase II: Non-Linear time-history analysis of the platform with added conductors

The analyses were carried out with 18 well conductors added (see Fig. 9). As in the prior phase, the time-history analyses were conducted for all 12 base cases. The results of these analyses are presented below.

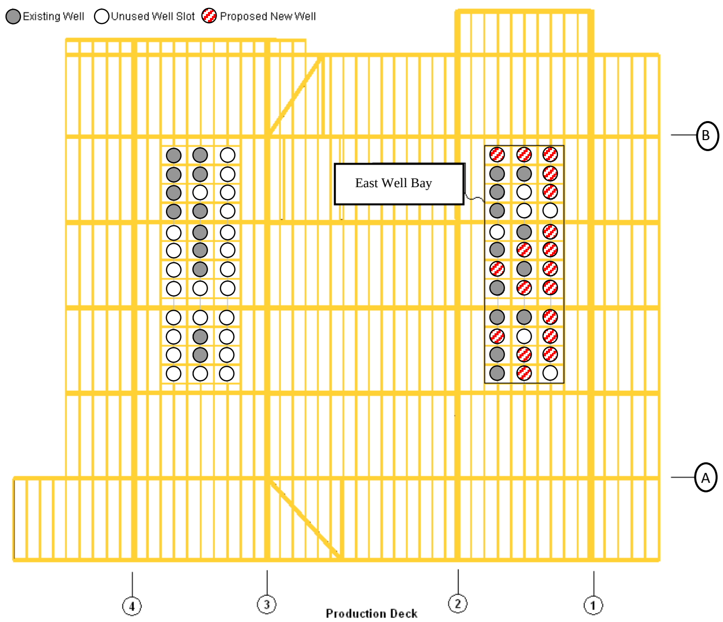


Fig. 9- Location of Additional well conductors- Plan View

Displacement: The maximum horizontal transient motion of the main deck is about 33 inches absolute and the base of the platform experiences horizontal absolute motion of about 24 inches. The maximum transient vertical movement of the platform, measured at its main deck, is about 7 inches. The horizontal permanent set of the deck, after the platform comes to rest, is 4.3 inches. The deck elevation is 2.3 inches lower than before the earthquake.

Pile structural behavior: The analyses indicate that local yielding of piles occurs during the maximum excursion cycles. A full plastic hinge did not form in the piles, with the exception of pile B1, where one full plastic hinge formed at 48 feet below mudline for the HE liquefied motions only; therefore a mechanism will not form that leads to collapse of the platform.

Soil demands: The highest loading of the soil in compression (8,000 kips) occurs at pile B4 during the HE earthquake motion. This load exceeds the pile's friction capacity, causing the maximum transient downward movement of the pile. However, in all cases studied the stored energy in the platform causes a rebound and the pile returns upward and at the end of the earthquake, and remains just several inches below its initial elevation.

4.4.3. Phase III: Reserve capacity of the platform with additional conductors

The reserve capacity assessment was carried out for the purpose of confirming that the structure is not precariously close to collapse if soil strength is lower, or if the intensity of the earthquake is higher than that used for the 1,000 year structural assessment. Lower bound (LB) and upper bound (UB) variations from the best estimate soil profile were used. The results of the analyses in Phase I and II indicate that the HE motion is the controlling motion. Therefore, the variation in earthquake intensity was considered by increasing the HE motion by 20% and 40%. This was achieved by simply factoring up the applied displacement time-histories, and repeating the time-history analysis for these two variations.

Displacements: The results show that the respective displacements are approximately 30% higher than those in Phase II. Table 4 presents a comparison of the maximum response obtained among the Phase II base case analyses with the maximum obtained from the Phase III reserve capacity analyses.

Pile structural behavior: The analyses indicate that local yielding of piles occurs during the maximum deck excursion cycles. Yielding occurs in all piles except B3, mostly between 30 and 65 feet below mudline (see Fig. 10). Plastic hinges form at 41 feet below mudline in piles A1 and B1 for the increased motions under non-liquefied conditions. The 40% increase in motion was also run for liquefied conditions. In this case, plastic hinges form in all piles except pile B3 near 41 feet below mudline. Plastic hinging also forms at mudline in piles B3 and A3. No full plastic hinge formation occurs for the LB and UB soil cases.

Table 4- Absolute Displacement Summary

DISPLACEMENT PARAMETER (inches)	Phase II, Max	Phase III, Max
Transient Horizontal at Deck Level	33.3	43.9
Transient Vertical at Deck Level	-6.6	-8.8
Residual Horizontal at Deck Level	4.3	5.6
Residual Vertical at Deck Level	-2.3	-4.9
Transient Horizontal at Pile Head	24.5	34.0
Transient Vertical at Pile Head	-6.0	-8.0
Residual Horizontal at Pile Head	1.6	4.1
Residual Vertical at Pile Head	-1.9	-4.4

Soil demands: The highest loading of the soil in compression (8,625 kips) occurs at pile B4 during the 140% HE earthquake motion. This load exceeds the pile's friction capacity, causing the maximum transient downward movement of the pile as indicated in Table 5. However, as shown in the table, in all cases studied the stored energy in the platform causes a rebound and the pile returns upward and at the end of the earthquake, remains just several inches below its initial elevation. The maximum demand/capacity ratio in uplift in any pile occurs in pile A1 (D/C=1.0) for the 140% HE (liquefied condition) motion. This uplift exceeds the pile's skin friction capacity and reaches the soil uplift capacity, which includes the suction capacity. A time-history plot of the corner pile axial force demand /capacity (D/C) ratio is provided in Fig. 11 for the 140% HE230 motion. The foundation redundancy of the 8-pile jacket configuration means that reserve capacity exists after the highest loaded piles reach their static capacity.

Table 5- Summary of Pile Peak Tensile and Compression D/C Ratios

Case	**A1**	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**
HE-UB	-0.83* 1.00*	-0.64* 0.45	-0.64* 0.29	-0.52 0.27	-0.61* 0.25	-0.65* 0.32	-0.56 0.36	-0.81* 0.90
HE-LB	-0.98* 1.00*	-0.98* 0.20	-0.98* 0.27	-0.98* 0.39	-0.99* 0.49	-0.98* 0.35	-0.95* 0.10	-0.97* 0.81
HE-120%	-0.85* 1.00*	-0.73* 0.51	-0.73* 0.38	-0.71* 0.17	-0.74* 0.25	-0.75* 0.48	-0.72* 0.29	-0.89* 0.96*
HE-140%	-0.89* 1.00*	-0.76* 0.60	-0.75* 0.48	-0.77* 0.33	-0.78* 0.42	-0.78* 0.55	-0.76* 0.31	-0.91* 0.98*
HE-lq-140%	-0.81* 1.00*	-0.70* 0.39	-0.71* 0.46	-0.77* 0.44	-0.77* 0.46	-0.73* 0.31	-0.70* 0.11	-0.87* 0.80

* Pile skin friction capacity is exceeded

Note: Uplift D/C is positive and compression D/C is negative.

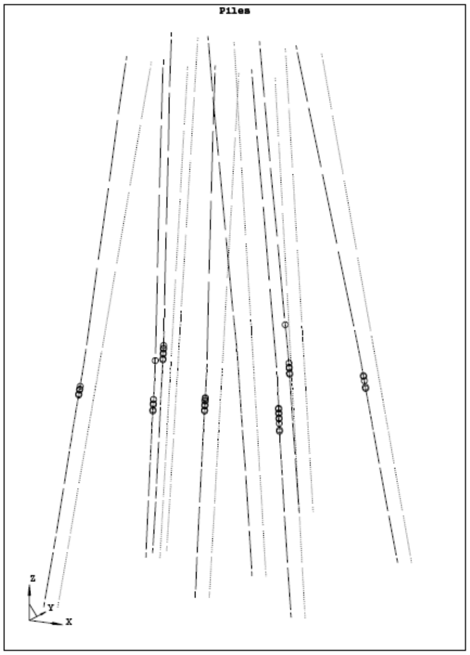


Fig. 10. Yield Locations in Piles, HE Motion, No Liquefaction, FM Model

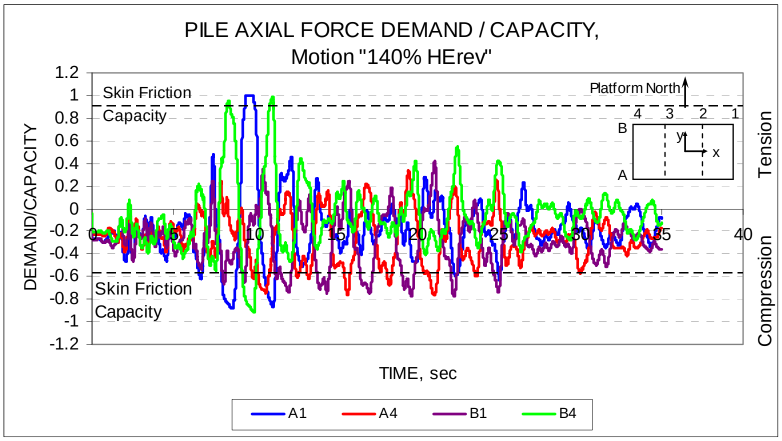


Fig. 11- Time-History of the Pile Axial Force D/C Ratio for the 4 Corner Piles, HE No Liquefaction

5. Conclusions

Based on this comprehensive set of analyses, we conclude that the platform, with the proposed additional conductors is not close to collapse as a result of the modern predictions of 1,000 year earthquake motions. The redundancy of the foundation system allows for sufficient reserve capacity so that the platform is not considered precariously close to collapse should the motions be higher or the soils weaker.

From studying the animated motions obtained from several of the time-history analyses, it is apparent that the platform's maximum displacements under the DLE are a response to one or two pulses. The platform undergoes a full cycle (plus and minus amplitude) at maximum excursion, but recovers from these peak displacements. Displacements in subsequent cycles are reduced, that is, somewhat proportional to the subsequent ground motion cycles. It is concluded that the platform responds to the ground motions as if to an impulse type loading rather than in a resonant fashion. The peak absolute horizontal resultant displacement at deck level, occurring during the first pulse of the controlling ground motion, is approximately 2.8 feet. The maximum horizontal resultant displacement at deck level is on the order 4 inches at the end of the shaking, whereas vertical displacement is between 2 to 4 inches downward.

The analyses indicate that local yielding of some piles occurs during the maximum excursion cycle. Yielding occurred in all but one pile, generally at depths approximately 30 feet below mudline, for non-liquefied conditions. Two inner piles also yield at mudline under liquefied conditions as well as for motion HE under non-liquefied conditions. Full plastic hinges did not form in the piles, with the exception of one pile, where a full plastic hinge formed at a depth of 48 feet below mudline for the HE liquefied motions only. Thus a mechanism does not occur which would lead to collapse of the platform.

The platform satisfies the requirements stipulated in Section 17 of API (Ref. 1), in that the platform has some reserve capacity to prevent collapse during this earthquake. Its reserve strength ratio has been estimated at 1.3. This reserve strength ratio was estimated by carrying out a ductility (pushover) analysis. It should be noted that increasing a 1000-year return period spectrum by a factor of 1.3 essentially results in ground motion inputs that have a return period on the order of 2,000 to 2,500 years.

6. References

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