EARTHQUAKE EFFECTS ON THE CANTILEVER BEAM OF A DRILLING JACKUP

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ABSTRACT
When carrying out site specific assessments for jack-ups to the International Standard ISO 19905-1, it is common practice to ignore local load effects, which are addressed during the design. The determination of whether a jackup is suitable for a specific location usually involves comparing the global structure of the jackup and the foundation loads to their respective capacities. Thus, for the traditional SSA, the strength of the cantilever beam and its support structure (push-up and hold-down) is not considered. While this assumption has proven valid for the storm condition assessment, further investigation is needed in regards to the effects of ground accelerations (earthquake loading) on the cantilever beam especially considering the potential loss of life, damage to surrounding assets, and environmental impact any failure to the cantilever might have. This paper explores the effects of earthquake loads on the cantilever beam of a representative drilling jackup. The resulting demands on the cantilever are assessed and compared to the demands on the global structure of the jackup.

KEY WORDS: Jackups, Elevated Assessment, Site Specific Assessment, Earthquake Analysis, Response Spectrum, Time History, Push-Up, Hold-Down, Cantilever Beam

INTRODUCTION

Seismic screenings of jackups have garnered more and more interest recently, coinciding with the increased operation of jackups in seismically sensitive regions. Typical seismic screenings have focused on a global assessment of the jackup. However, further investigation of the local structure, specifically the cantilever beam and its support structure is likely needed, since failure of the cantilever beam presents a similar level of consequence in regards to life-safety, damage to the environment, and anticipated losses as does a global failure of the jackup. This paper looks to compare the degree of utilization of the critical members of the cantilever beam (push-up and hold-down) with the critical members of the jackup (chords, braces, etc.) when subject to seismic loads. The comparison is to assess the potential risks to the cantilever beam structure in relation to the global jackup structure using a detailed mass model of a generic jackup and the assessment methodology provided in ISO 19905-1 [1] and ISO 19901-2 [2].

BACKGROUND

When carrying out site specific assessments (SSAs) for jackups to the international standard ISO 19905-1 [1], it is common practice to ignore local load effects, which are addressed during the design. The determination of whether a jackup is suitable for a specific location usually involves comparing the calculated structural loadings on the holding system, leg members (chords and braces) and spudcan reactions against their capacities. Thus, for the traditional SSA, the strength of the cantilever beam typically installed on most drilling jackups, or its supporting structures (push-up and hold-down) is not considered.

While this assumption has proven valid for the storm condition assessment, further investigation is needed in regards to the effects of ground accelerations (earthquake loading) on the cantilever beam. This is especially true when considering that the natural periods for the primary cantilever modes tend to fall in the range of the excitation periods typically experienced from an earthquake.

The cantilever beam is a determinate structure and thus has no redundancy in regards to its supports. It also has an exposure level that is on par with the entire jackup itself, considering the high concentration of personnel, direct connection to the well through the drill string, and large capital investment due to the sophisticated drilling equipment typically contained on or within it. Each of these points highlights the need for the cantilever beam (and its supports) to be investigated in combination with the global effects of the jackup.

Published literature on the vulnerabilities of the cantilever beam is lacking. Several recent studies on the assessment of jackups for seismic loading have been documented. One of these recent studies by Elsayed et al., 2014 [3] focused on the use of an incremental dynamic analysis or pushover analyses, for evaluating the

performance of a jackup subjected to seismic loading. The study focused solely on the global response of the jackup, without providing specific attention to the cantilever beam.

A recent study by Stock et al., 2015 [4] did account for the cantilever beam in the finite element model used in their analyses. However, the inclusion of the cantilever beam was primarily for proper mass modeling and resulting stresses or push-up/hold-down loads were not part of the reported results, as the intention of the study was to evaluate the general demands on the jackup, specifically the foundation reactions.

The authors note that providing generic guidelines on the severity of the cantilever beam to the overall jackup is difficult, considering that designs for drilling jackups and their respective cantilever beams vary widely. To be sure, the results and observations presented in this paper are applicable only to the very specific, fictitious (albeit realistic) jackup/cantilever beam design selected by the authors. Thus, this study is only intended to provide evidence to the demands on the cantilever for severe earthquake loading in comparison to the global structure of the jackup, and shed light on the possibility that for areas with seismic activity, it may not be enough to check the jackup from a global perspective.

JACKUP AND CANTILEVER BEAM PARTICULARS

For the purposes of this study, a generic jackup suitable for working in severe environmental conditions in water depths up to 300 ft was used. The jackup is considered representative of the average properties of multiple existing rig designs and is assumed to have tangential guides and utilize a fixation system in the elevated condition. The particulars of the jackup are given in Table 1. The details for the chords and braces are given in Table 2, with a split-tube type cross-section assumed for the chord.

Table 1 Jackup Particulars

Component	**Value**
Hull Length	212.1 ft
Hull Breadth	200.5 ft
Hull Depth	26.0 ft
Hull Elevated Weight	15,000 kips
Operating VDL	6,000 kips
Transverse Leg Spacing	140 ft
Longitudinal Leg Spacing	129 ft
Leg Length	411 ft
# of Chords Per Leg	4
Chord Spacing	31.5 ft
Spudcan Height	20 ft
Effective Spudcan Diameter	45 ft
Total Leg Weight (all 3, and including Spudcans)	7,000 kips
Total Spudcan Weight (all 3)	1,500 kips
Spudcan Volume	10,000 ft^3
Fixation System Stiffness (per Chord)	20,000 kips/in
Preload Reaction	15,500 kips

Table 2 Details of Chord and Brace Members

Component	Chord	0.75" WT Brace	0.5" WT Brace
Area	150 in^2	28.3 in^2	19.2 in^2
I_{y-y}	3,995 in^4	511 in^4	362 in^4
I_{z-z}	3,653 in^4	511 in^4	362 in^4
F_y	100 ksi	85 ksi	85 ksi
k-factor	1.0	0.7	0.7

The cantilever beam was assumed to have a maximum reach of 45 feet for the center of the rotary, aft of transom. This cantilever reach is representative of older technology, but a design that is still utilized quite frequently today. The cantilever beam is designed to have a static deflection when fully loaded at the maximum extension of L/240, or approximately 2.25 inches at the center of the rotary.

Table 3 Details of the Cantilever

Component	Value
Max Cantilever Reach (rotary center aft of transom)	45 ft
Max Hook/Setback/Rotary Loads	1,250 kips
Beam Depth	13.75 ft
Cantilever Beam (Per Beam) I_{x-x}	2,276,000 in^4
Cantilever Beam (Per Beam) I_{y-y}	13,900 in^4
Push Up/Hold Down Spacing	39.0 ft

As conductor tension and hook loads are not associated with masses, they are not considered in the mass model [1]. Thus, the highest loading condition can be considered to be the pipe hanging from the rotary or in the setback. The drilling load was assumed to be 450 kips in the setback and a rotary load of 800 kips.

WEIGHTS AND ADDED MASS

Only the critical load case was considered for this analysis (i.e., with the cantilever fully loaded at maximum reach). It is assumed that significant reserve strength is present when the cantilever is not fully extended and thus damage to the cantilever or its support structure from seismic loading while in a position short of maximum is unlikely.

The overall center of gravity ("CG") of the elevated weight was assumed to be positioned at the geometric center of all three legs as this is typically considered best practice in the industry (although noted to not always be realistic). The position of the VDL was adjusted to achieve this overall CG even with the cantilever fully extended.

The weights and center of gravity values used for the analysis are given in Table 4. The leg is assumed to be fully retracted with the spudcan tip in line with the keel. The origin of the reported CG values is assumed to be as follows:

- LCG: Frame 0 (Bow), +AFT
- TCG: Centerline, +STBD
- VCG: Keel, +Up

Table 4 Weight and CG Values

Component	Weight	LCG	TCG	VCG
	kips	ft	ft	ft
Hull Lightship	13,225	113.6	0.0	23.8
Cantilever Beams/Pipe Rack	900	225.2	0.0	36.2
Substructure/Drill Floor/Equipment	875	261.7	0.0	80.6
VDL (except Drilling Load)	4,750	37.0	0.0	13.0
Combined Drilling Load	1,250	243.4	0.0	80.0
Total Elevated Weight	**21,000**	**115.0**	**0.0**	**27.6**
Leg Weight (All 3 Legs)	5,500.0	115.0	0.0	215
Spudcan Weight (All 3 SCs)	1,500.0	115.0	0.0	10
Total Leg Weight (All 3 Legs)	**7,000.0**	**115.0**	**0.0**	**171.1**

Added mass was considered for both the submerged portion of the legs and the spudcans. As per Stock et al. [4], the spudcan added mass was assumed to be equal to twice the water volume contained in the spudcan or 1,280 kips.

SITE CHARACTERISTICS

A single site was chosen for the analysis. The site has a shallow water depth and stiff foundation leading to relatively low natural periods for the jackup (likely to result in more severe loading from the earthquake ground motions). The particulars for the assumed site are shown in Table 5.

Table 5 Site Characteristics

Water Depth	75 ft
Penetration	10 ft
Air Gap	45 ft
Soil Type	Medium-Dense Sand
Site Characterization	Stiff
Soil Mass Density	125 pcf
Soil Shear Modulus, G	955 ksf
Soil Poisson's Ratio, ν	0.35
Soil Vertical Stiffness, k_3	1.32E+08 lbs/ft
Soil Horizontal Stiffness, $k_1 = k_2$	1.06E+08 lbs/ft
Soil Rotational Stiffness, $k_4 = k_5$	4.47E+10 lbs*ft/rad

FINITE ELEMENT MODEL

A detailed finite element model of the jackup and cantilever beam was created in SAP2000 Ultimate (v19.1.0), a commercially available structural analysis program. The hull was modeled using shell elements and all major bulkheads and decks were included. Stiffeners were also included in the model to avoid unrealistic local deflections of the bulkheads and decks. The jackcases and jackcase bracing were modeled with beam elements with the appropriate equivalent stiffness.

The truss legs were constructed of prismatic members representing the chords with proper member orientation and tubular members for the braces and spanbreakers. The fixation system, upper guides and lower guides were represented by beam elements connected to the leg and jackcase with the appropriate member-end-releases. The cantilever beam was modeled in detail using shell elements. The drill floor support structure and drill floor were also modeled in detail. Beam elements were used to represent the model for the derrick. An isometric view of the complete finite element model is shown in Figure 1.

All weights were applied as distributed mass in the model. The mass of the drill pipe in the setback was modeled by increasing the mass of the derrick. The rotary load was accounted for by increasing the mass of the drill floor beams. The spudcan added mass was applied as a joint load at the horizontal reaction point.

To model the push-up and hold-down, a series of link elements were used to connect the cantilever beam to the hull structure. A close-up view of the cantilever and its connection to the hull (with the legs, jackcases, and hull stiffeners removed for clarity) is shown in Figure 2.

A compression-only element is preferred for the link representing the push-up connection. However, because a response spectrum analysis is performed in the frequency domain it is not possible to include a nonlinear compression-only element for the link. So, a link element capable of taking both tension and compression was used to model the push-up for all analyses for consistency. This is considered a non-issue, as the results show the link elements to remain in compression throughout all time history analyses.

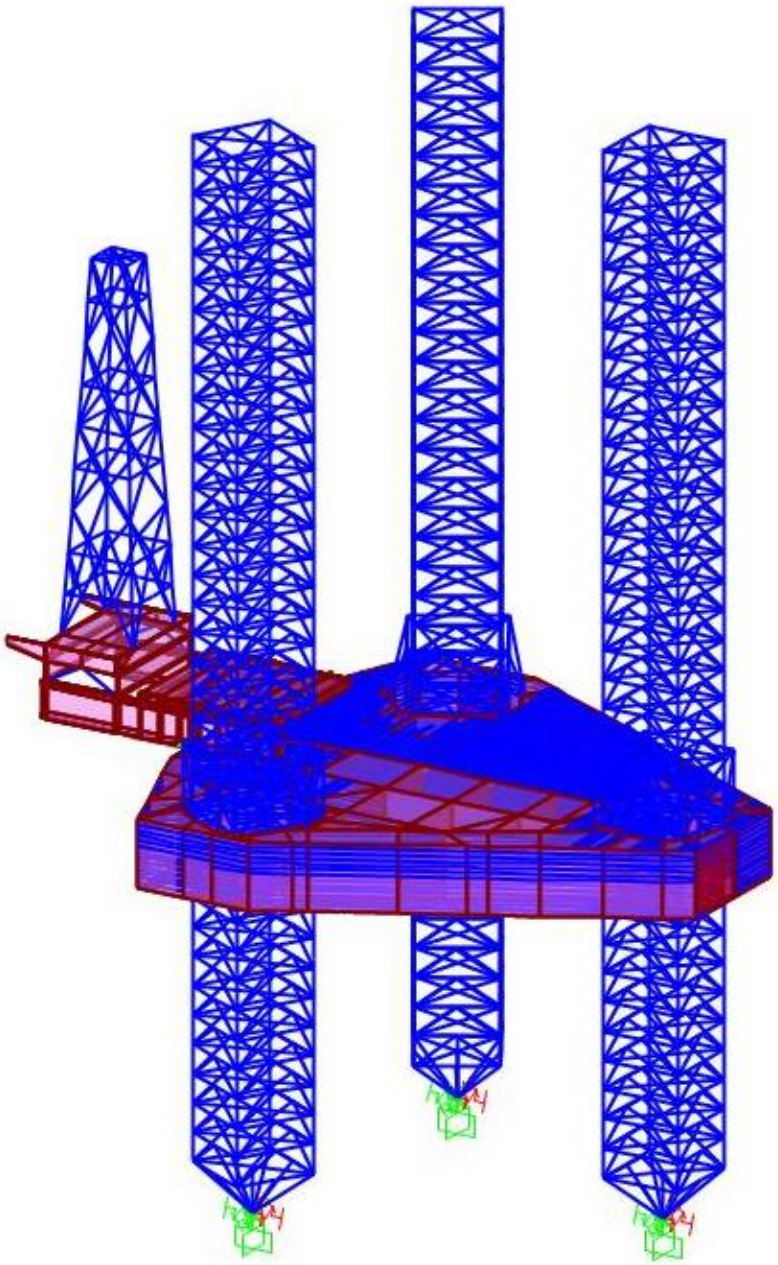

Figure 1 Complete Finite Element Model

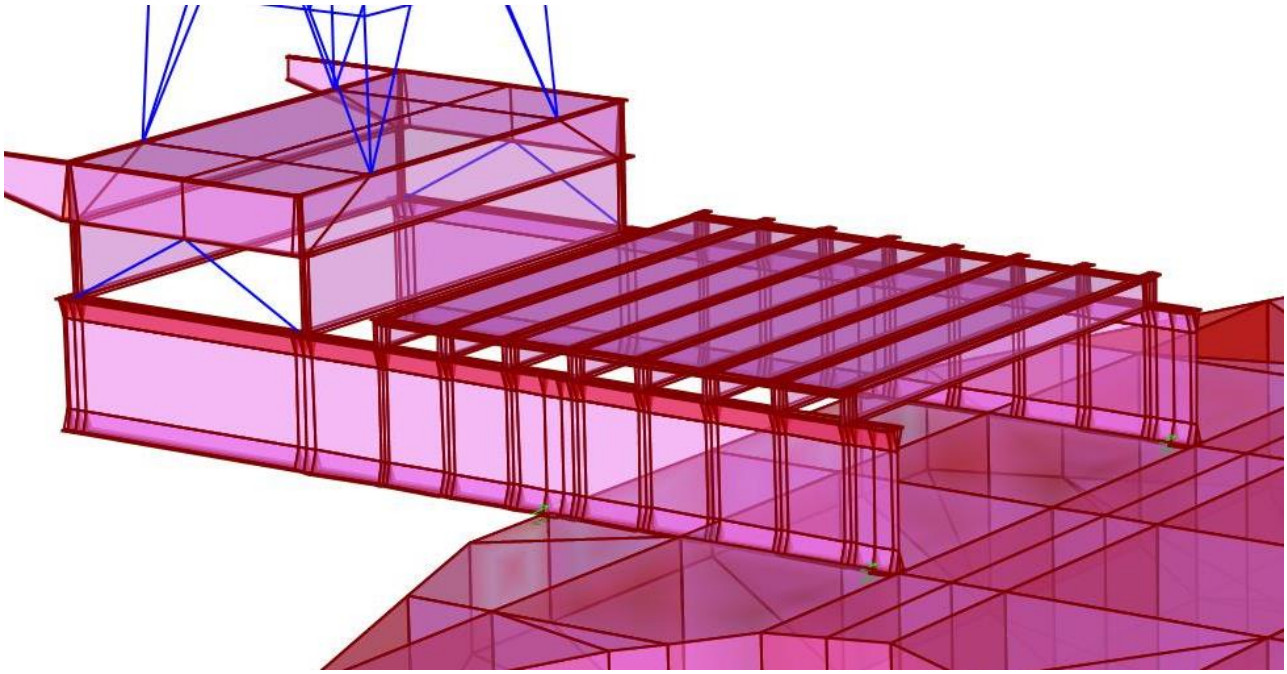

Figure 2 Cantilever Beam Close-up

EARTHQUAKE SPECTRA AND TIME HISTORIES

For the seismic analyses performed for this paper, two methodologies were used - response spectrum analysis and time history analysis. For the response spectrum analysis, accelerations at the upper end of Zones 2 and 3 were analyzed. As noted previously, the soil was assumed to be a medium-dense sand, typically described as stiff to very stiff soil, resulting in a site class of D. The particulars for each seismic zone analyzed are given in the following Table.

Table 6 Seismic Zone Particulars

Site Seismic Zone	2	3
Sa,map(0.2)	0.625	1.125
Sa,map(1.0)	0.25	0.45
Site Class	D	D
C_a	1.3	1.05
C_v	1.9	1.55

The ISO 19902-1 [2] methodology was employed to develop the two response spectra from the parameters chosen for a 1,000-year event. The resulting horizontal spectra are given in Figure 3. The vertical spectra for each seismic zone were assumed to be 50% of the horizontal spectra.

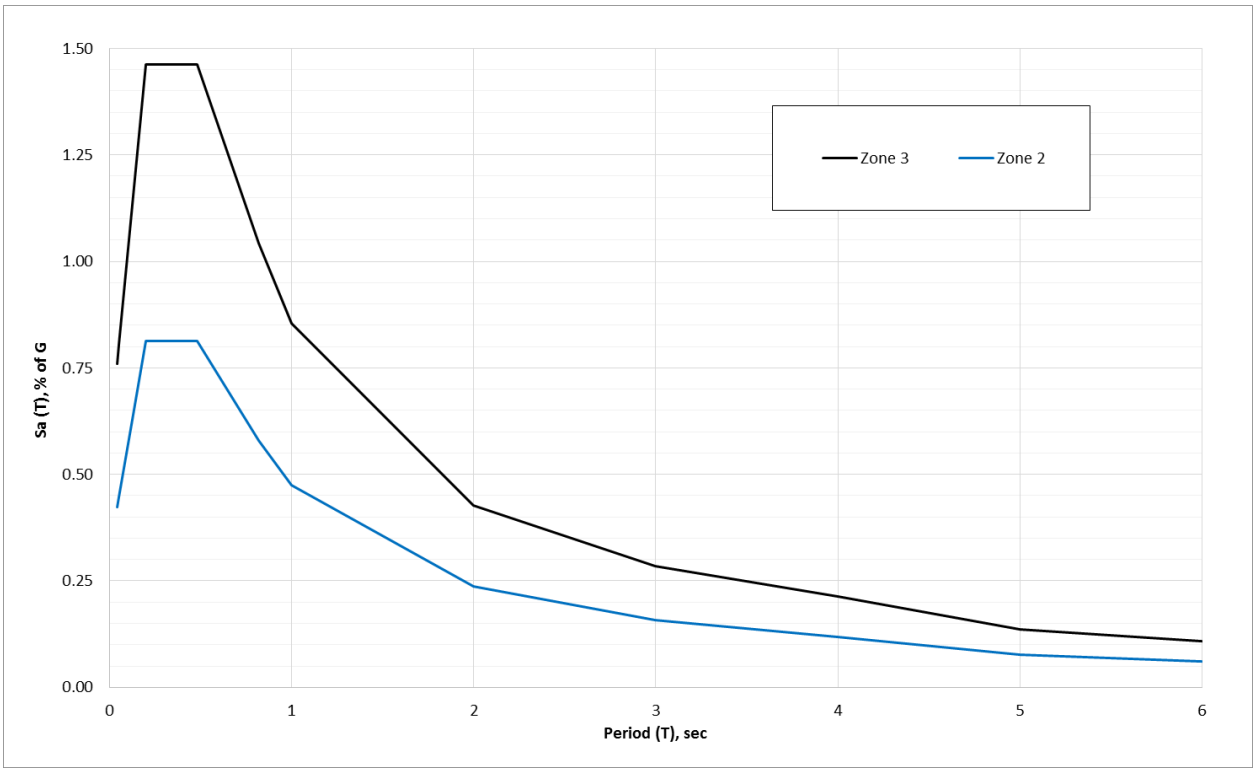

Figure 3 1,000-Year Response Spectra (5% Damping)

For the time history analysis method, four (4) sets of earthquake ground motions that resulted in a mean response spectrum that fell approximately between the Zone 2 and 3 response spectra of Figure 3 were chosen. The particulars of each ground motion record are given in Table 7. The soil shear wave velocity for each earthquake was kept between 180 and 350 m/s, as noted for a Site Class of D in ISO 19902-1.

Table 7 Earthquake Time Histories

Earthquake Name	Year	Magnitude	Mechanism	Rrup (km)	Vs30 (m/sec)	5-75% Duration (sec)	5-95% Duration (sec)
Dinar_ Turkey	1995	6.40	Normal	3.36	219.8	10.3	16.7
Imperial Valley-06	1979	6.53	strike slip	3.95	205.6	3.8	9.6
Darfield_ New Zealand	2010	7.00	strike slip	1.22	344.0	5.6	16.0
Duzce_ Turkey	1999	7.14	strike slip	6.58	281.9	7.3	11.1

Recorded ground motions were taken from the PEER Ground Motion Database NGA-West 2 [8]. A plot of the resulting arithmetic mean (and +/- 1 standard deviation) of the response spectra for the four ground motion records for the two horizontal directions is given in Figure 4. A similar plot of the vertical accelerations is given in Figure 5. The response spectra for Zone 2 and Zone 3 are provided in each plot for reference. The reader should note that the response spectrum for Zones 2 and 3 is to be applied to both the X and Y horizontal directions. Thus, comparison of both the H1 and H2 accelerations to both response spectrum curves is applicable in Figure 4.

As is evident from the plots, the mean spectrum for both the X and Y horizontal accelerations falls between the Zone 2 and Zone 3 plots. Also evident is a pronounced peak for the H2 + 1 Std Dev at an approximate period of 1.3 seconds. This peak is due in large part to the very high accelerations (~1.2Gs) seen for the Darfield time trace at approximately the same period.

For the vertical accelerations, the time history accelerations appear to drop below the Zone 2 accelerations for periods greater than 0.5 seconds. While the response spectrum from the time history values is significantly

higher for the lower periods, it is not expected that these periods will result in the excitation of any major structure on the jackup.

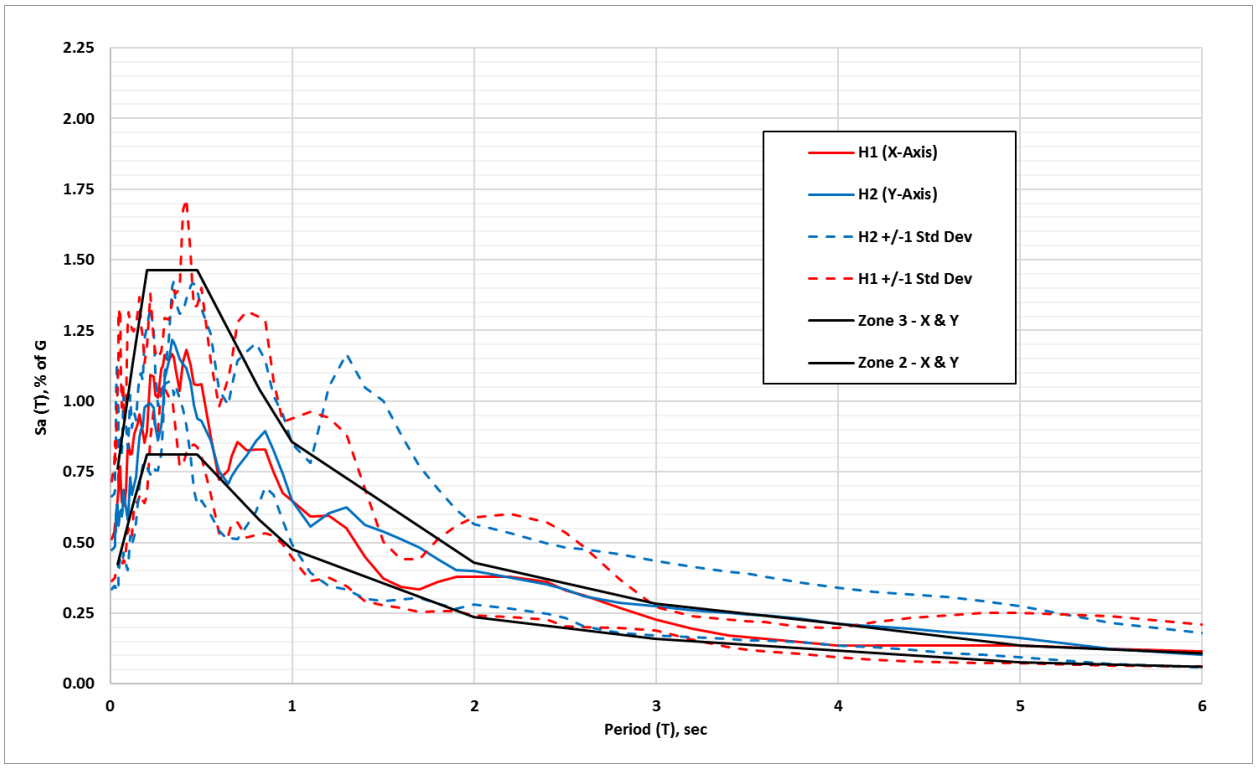

Figure 4 Response Spectra for Horizontal Accelerations

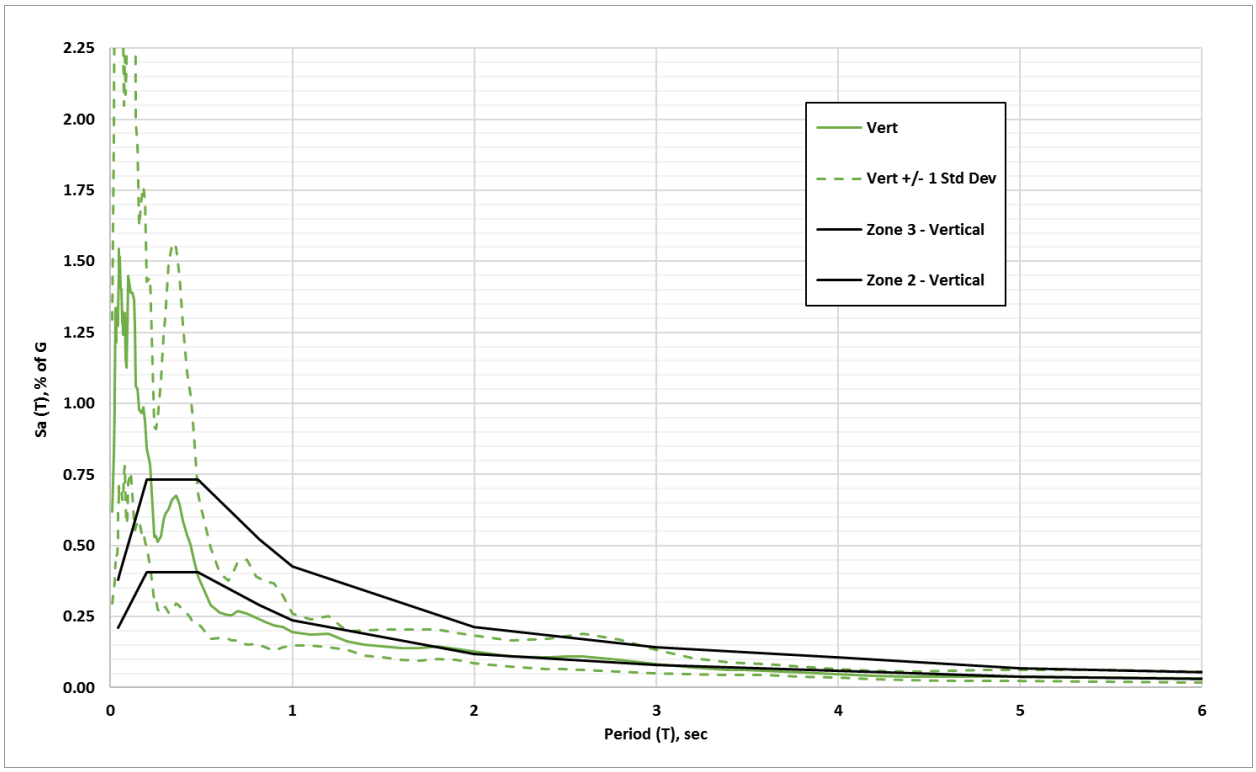

Figure 5 Response Spectra for Vertical Accelerations

STATIC ANALYSIS

To establish the criticality of the cantilever in relation to the global jackup, the design parameters for the cantilever must be established. Thus, a static analysis was performed with the cantilever at maximum extension with the full drilling load to determine the resulting static push-up and hold-down reactions.

The resulting reactions for each push-up and each hold-down for the static analysis were found to be 2,950 kips and 1,438 kips, respectively. Assuming the structure for the push-up and hold-down was designed following a Working Stress Design approach [5], it can reasonably be assumed that these values would result in stresses of approximately 60% of yield. Also, considering the structure is likely built from ABS grade hull steel [7], the ultimate strength of the material would be on average 57% higher than yield. From the baseline values established for the push-up and hold-down reactions from the static analysis, the resulting push-up and hold-down reactions that would lead to the material reaching its yield and ultimate stress can be reasonably calculated and are given in Table 8. These values are utilized to classify the criticality of the cantilever structure when subjected to seismic loading.

Table 8 Push-up and Hold-down Limiting Values

Component	Static Load	Load at Initial Yield	Load at Failure
	kips	kips	kips
Push-Up	2,950	4,917	7,719
Hold-Down	1,438	2,396	3,761

EARTHQUAKE ANALYSIS METHODOLOGY

Analyses based on the response spectrum and time history methodologies were completed for the jackup utilizing the conditions presented in Tables 6 and 7. The base excitations for both sets of analyses consisted of the two horizontal motions and the vertical motion.

For the response spectrum analysis, the vertical accelerations were assumed to be half of the corresponding horizontal acceleration values [2]. The horizontal accelerations were assumed to act in both the X- and Y-directions (simultaneously). The modal responses were combined using the complete quadratic combination (CQC) method and the directional accelerations were combined using the square root of the sum of the squares (SRSS) method as per API RP 2A [10].

For the time history analysis, independent acceleration time histories for all 3 directions (as acquired from the PEER database [8]) were applied to the model simultaneously. The time history analysis cases were run using the direct integration solution type. Each simulation was run for a sufficient length of time to capture the significant accelerations from the recorded data provided from the PEER database [8]. An initial 1.5-sec ramping segment for application of the dead load was used at the beginning of each simulation. The total length of each time history simulation was as follows: Darfield: 50 sec, Dinar: 35 sec, Duzce: 30 sec, Imperial Valley: 35 sec. A time step of 0.005 seconds was used.

For both the frequency-domain and time-domain analyses, radiation damping of the vertical modes was considered as per ISO 19905-1 [1]. For the linear modal dynamic analyses, the calculated contribution to the linear damping ratio in the vertical mode was found to be 9.8%. The radiation damping coefficient for the vertical dashpot in the linear time-domain dynamic analyses was found to be 23,367 kips/(ft/s).

Noted issues exist with the calculation for the vertical foundation radiation damping using the equations given in ISO 19905-1 [1]. The determination of the proper calculation of these values is beyond the scope of this paper and thus the published equations in ISO 19905-1 [1] were utilized which means the damping in the frequency-domain is likely too small. Stock et al. [4] provide further discussion of this issue.

As noted in Stock et al [4], whether P-delta effects are to be included in the seismic analyses is not clearly stated in the standard. While P-delta effects should be included in all assessment models, irrespective of the analysis being performed, for a model of this size the inclusion of P-delta in the transient analysis resulted in an unstable model as material nonlinearity is required to be included if P-delta is included in SAP2000. Thus,

for all analyses performed in this study, P-delta effects and material nonlinearity were not considered. While both effects should be considered for a proper assessment, for this study the exclusion of both is considered acceptable as the intent is to compare the demands on the cantilever to those of the entire jackup structure and both will be subjected to the same actions.

A total of 100% of the small strain spudcan-soil translational and rotational stiffness values were applied in all analyses. All partial action factors were assumed to be 1.0.

To determine the relative demands on the cantilever structure, the push-up and hold-down reactions were determined from each analysis. ISO 19901-2 [2] states that, "During the ELE event, structural members and foundation components are permitted to sustain localized and limited non-linear behaviour (e.g. yielding in steel, tensile cracking in concrete)." Thus, to determine a utilization ratio for the push-up and hold-down reactions, the ratio of the resulting reactions to the assumed load at initial yield established in Table 8 was taken.

The derrick was also taken into consideration when reviewing the relative demands on the cantilever. Derrick lateral accelerations were taken at the approximate location of the vertical CG of the derrick (45% of derrick height). A generic limiting design acceleration is assumed to be 0.5g's of lateral acceleration. A value of 0.5g is conservatively what a derrick on a jackup might experience in the ocean transit condition for pitch and roll motions of 15 degrees at 10 seconds [11]. The utilization ratio for the derrick is taken as the ratio of the lateral acceleration of the derrick compared to the limiting design acceleration. The API 4F [12] standard allows only for a 1/3 increase in the allowable values for the earthquake condition which is assumed to be present in the allowables for the design condition at 0.5g.

For quantifying the relative demands on the global structure of the jackup, the chord strength utilization, brace strength utilization, and bearing capacity utilization were all considered. The chord and brace strength utilization ratios were calculated using the code checking capabilities of SAP2000 following the LRFD approach set forth in AISC [6]. These values have been shown to closely correlate with the values calculated using the leg strength assessment methodology set forth in ISO 19905-1 [1]. The bearing capacity utilization ratio is set as the ratio of the preload reaction (15,500 kips) to the maximum vertical reaction from the analysis.

EIGENSOLUTION

A total of 200 mode Eigensolutions were computed for use in the response spectrum analysis. The first, second and third translational modes are attributed to sway, surge, and yaw with periods ranging from 2.7 to 2.3 seconds. The vertical translational modes, which also resulted in significant excitation of the cantilever, were found to be modes 20 and 21. Table 9 summarizes the results of the Eigensolutions.

For the 200 modes, the static mass utilization was approximately 99.9% for all directions. For the dynamic mass, which includes any added mass in the model, only 96.3% mass utilization was achieved for the vertical direction This is believed to be due to the spudcan added mass which is placed at the support and thus is likely excited in the vertical direction at an extremely high frequency, one that is not present in the 200 modes analyzed. As the Uniform Building Code [9] notes that a minimum mass utilization of 90% be achieved in a response spectrum analysis, the values achieved for the 200 modes in this analysis are considered acceptable. The dynamic mass utilization ratios for an increasing number of modes is shown in Table 10.

Table 9 Description of Pertinent Modes

Description of Modes		
Mode	**Period (sec)**	**Description**
1	2.68	Sway
2	2.36	Surge
3	2.33	Yaw
20-21	0.53 - 0.54	Vertical

Table 10 Mass Participation Values

Mass Participation			
# of Modes	**X**	**Y**	**Z**
10	96.30%	93.70%	2.30%
50	96.50%	96.20%	87.70%
100	96.60%	97.40%	95.60%
200	99.90%	99.90%	96.30%

A local model of just the cantilever beam and link support elements, as shown in Figure 6, was created to determine the specific modes for the cantilever. Table 11 details the main modes and their periods for the cantilever.

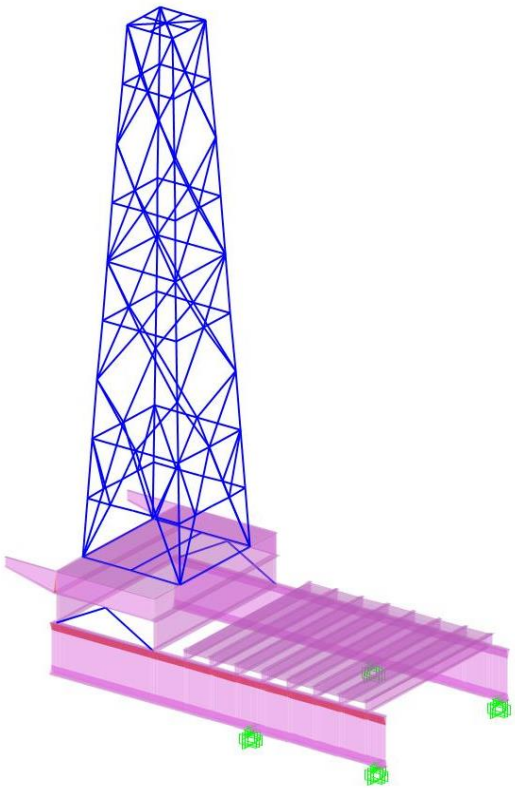

Figure 6 Cantilever Only Model

Table 11 Cantilever Modes

Mode	Period (sec)	Description
1	2.02	Bending about minor axis (sway)
2	0.96	Bending about major axis (vertical)
3	0.79	Torsion (bending about axial direction)

RESULTS

The resulting total base reactions on all three legs for each case analyzed are given in Table 12. As is evident from the reactions, the horizontal directions (X & Y) for each of the time histories fall in the range of the response spectrum values, but the variance between the response spectrum and time history analyses for the vertical reactions (Z) is quite high, noting that the maximum vertical reaction for Zone 2 is greater than any of the maximum vertical reactions for the time history analyses.

The resulting maximum and minimum vertical reactions on any one individual footing are also provided for reference. As is evident from the results provided, uplift was seen (negative values for the minimum vertical reactions) for both the Zone 3 response spectrum analysis and the Darfield time trace.

Included in Table 12 are the resulting maximum push-up and hold-down reactions, and the maximum derrick acceleration values as a percentage of gravity. Clearly, the resulting values for the push-up reaction for the response spectrum analyses exceed those from the time history analyses. Also evident is uplift at the push-ups for both the Zone 2 and 3 response spectrum analyses (the link connection is in tension).

As ISO 19901-2 [2] notes that the time history analysis method is recommended for obtaining relative deck motions, the results of this study also confirm this should be applied to the analysis of the cantilever as the response spectrum method likely overestimates the response of the cantilever to seismic loading. Two caveats to this statement are the issues with radiation damping discussed previously and the response spectrum analyses had greater vertical accelerations at the periods of excitation of the cantilever than did the time history analyses. Thus, if one were to be analyzing an earthquake time trace with large vertical accelerations at greater periods or if more damping were considered in the response spectrum analysis, this would likely result in a closer match of the different analysis methodologies, however this still would not merit use of the response spectrum analysis over the time history analysis.

Table 12 Total Base Reactions and Push-up/Hold-Down Loads

Analysis	Max/ Min	Base Reaction X	Base Reaction Y	Base Reaction Z	Max Vert. Reaction	Push-Up Load	Hold-down Load	Derrick Acc.
		kips	kips	kips	kips	kips	kips	%g
Response Spectrum Zone 2	Max	5,159	4,127	33,659	14,437	-5,905	2,270	0.62
	Min	-5,159	-4,127	22,341	4,495	4	605	
Response Spectrum Zone 3	Max	9,364	7,640	38,028	19,708	-8,246	2,950	1.12
	Min	-9,364	-7,640	17,595	-776	2,424	-72	
Darfield Time Trace	Max	13,580	8,906	31,069	19,286	-5,851	3,001	0.94
	Min	-8,605	-9,681	24,559	-1,126	-1,078	299	
Dinar Time Trace	Max	4,992	2,615	29,506	13,335	-3,918	2,037	0.32
	Min	-5,440	-2,468	26,668	5,781	-1,969	772	
Duzce Time Trace	Max	3,976	3,950	29,200	13,893	-3,889	1,804	0.38
	Min	-4,958	-7,257	26,479	4,658	-1,939	942	
Imperial Valley Time Trace	Max	5,201	9,633	30,194	15,133	-4,528	1,975	0.69
	Min	-6,042	-7,915	25,595	1,721	-2,103	960	

The utilization ratios for each component are given in Table 13. Due to the large base shear encountered for all analyses, the local bending on the chords from the guides is a significant factor in the calculation of the chord UC. To provide for the effect on the UC due to the positon of the guides relative to the chord-to-brace connection, two positions for the guides were evaluated: the lower guide positioned at a node (chord-to-brace connection) and the lower guide positioned at the approximate mid-point of a bay. Thus, two UC values are provided for both the chords and the braces, based on the guide position. No checks of the fixation system were performed, as it is assumed it will be designed to be stronger than the chords.

Table 13 Utilization Ratios

Analysis	Push-Up UC	Hold-Down UC	Derrick UC	Chord UC[1]	Brace UC[1]	Bearing UC
Response Spectrum Zone 2	1.20	0.95	1.24	0.55 / 0.80	0.58 / 0.63	0.93
Response Spectrum Zone 3	1.68	1.23	2.25	0.85 / 1.34	1.06 / 1.21	1.27
Darfield Time Trace	1.19	1.25	1.87	1.23 / 1.75	1.71 / 1.69	1.25
Dinar Time Trace	0.80	0.85	0.65	0.54 / 0.75	0.59 / 0.68	0.86
Duzce Time Trace	0.79	0.75	0.77	0.68 / 0.87	0.68 / 0.76	0.90
Imperial Valley Time Trace	0.92	0.82	1.39	0.84 / 1.09	1.02 / 1.07	0.98

[1]UC values are as follows: lower guide positioned at a brace-to-chord connection / lower guide positioned at the mid-point of a bay.

To provide further clarity on how the demands on the cantilever beam and derrick compare to the global demands on the jackup, the plot in Figure 7 is provided. The UCs related to the demands of the jackup are shown in shades of grey to draw a definitive contrast between the colored bars of the UCs related to the demands on the cantilever beam or derrick. As is clear from the plot and the results given in Table 13, UC values for the push-up are greater than all parameters checked for the jackup for the response spectrum analyses. However, the applicability of using a response spectrum analysis for determining the demands on the cantilever has been discussed previously in this paper.

Looking at the results of the time history analyses, the demands on the cantilever tend to fall in the general range as those for the jackup. For the Dinar time trace, the structural demands on the cantilever exceeded the demands on the leg structure for any guide position. For the Duzce and Imperial Valley time traces, when the guide is positioned at the node the chord structure has a lower UC value than the cantilever. It does appear that for the most severe ground motions, specifically the Darfield time trace, a larger demand is put on the global structure of the jackup than the cantilever. The Derrick UC values are seen to be greater than those on the cantilever beam for all cases, except for the Dinar and Duzce time trace.

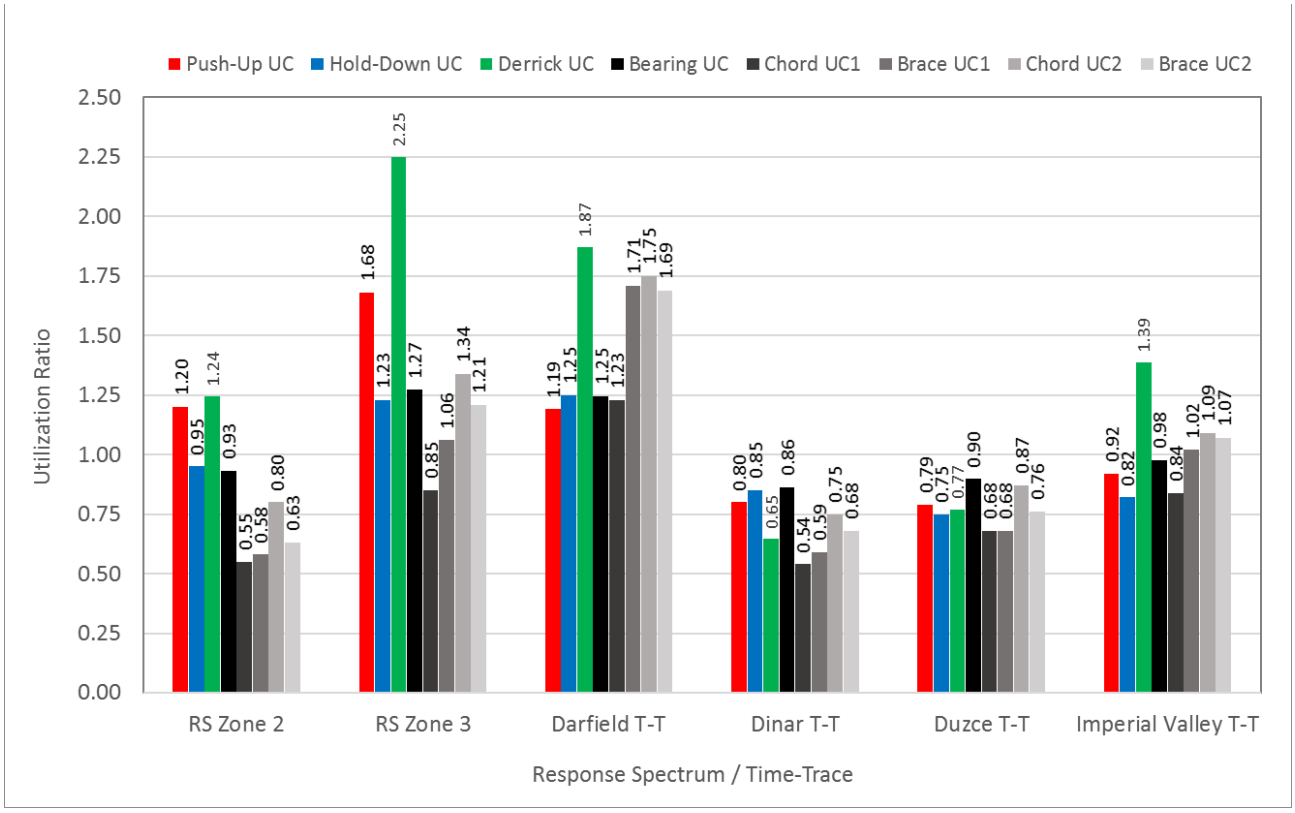

Figure 7 Earthquake Analysis Results

SUMMARY AND CONCLUSIONS

To better understand the relative demands on the cantilever beam in comparison to the global structure of a drilling jackup, both response spectrum and time history earthquake analyses were performed. The response spectrum analyses produced reactions on the push-up and hold-down that far exceeded those of any of the four time histories analyzed, reinforcing the fact that a time history analysis should be used in determining the response of the cantilever beam to seismic loads.

From the results of the time history analyses performed, the demands on the cantilever beam appear to be of similar magnitude to the demands on the global structure of the jackup with the results of this specific study showing that the more severe the earthquake ground motions, the greater the demands on the jackup structure as opposed to the cantilever. Further study is needed to determine if this is simple coincidence or an actual trend. In addition to the findings regarding the cantilever beam, the findings for the drilling derrick showed that for significant earthquake events the lateral accelerations for which the derrick is likely designed will be exceeded.

The authors note that these conclusions are limited to this specific and fictitious design, but that the generic nature of the model provides evidence that the cantilever structure and derrick should be evaluated in detail with the overall jackup structure in an earthquake analysis. Of specific concern are instances where the cantilever beam's natural periods at the extreme reach position are near the relevant periods of the seismic acceleration spectrum. Obviously, if the relative strength of the cantilever beam in relation to the jackup were to be skewed in either direction, then the importance of checking the cantilever beam would change, accordingly. Also, if it were verified that the derrick was designed for a certain level of earthquake, or significant lateral accelerations, as per API 4F [12], then a detailed analysis of the derrick may not be necessary.

FUTURE WORK

A model of this size and complexity, coupled with time domain analyses requires significant time and effort, both in man hours and computation time and thus the work described in this paper is admittedly limited in the number of cases and variations analyzed. Future work that is envisioned using this model includes:

1. Analyze additional cases at varying deployed leg lengths and cantilever beam designs.
2. Inclusion of p-delta effects in the time history analyses.
3. Incorporating a detailed connection with contact elements for the push-up and hold-down into the model.
4. Further study of the deflection of the cantilever and the effects this could have on the drill string, well, platform, etc.

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