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A Study on Load-carrying Capacity Design Criteria of Jack-up Rigs under Environmental Loading Conditions

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환경하중을 고려한 Jack-up rig의 내하력 설계 기준에 대한 연구

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Abstract : *Jack-up drilling rigs are widely used in the offshore oil and gas exploration industry. Although originally designed for use in shallow waters, trends in the energy industry have led to a growing demand for their use in deep sea and harsh environmental conditions. To extend the operating range of jack-up units, their design must be based on reliable analysis while eliminating excessive conservatism. In current industrial practice, jack-up drilling rigs are designed using the working(or allowable) stress design (WSD) method. Recently, classifications have been developed for specific regulations based on the load and resistance factor design (LRFD) method, which emphasises the reliability of the methods. This statistical method utilises the concept of limit state design and uses factored loads and resistance factors to account for uncertainly in the loads and computed strength of the leg components in a jack-up drilling rig. The key differences between the LRFD method and the WSD method must be identified to enable appropriate use of the LRFD method for designing jack-up rigs. Therefore, the aim of this study is to compare and quantitatively investigate the differences between actual jack-up lattice leg structures, which are designed by the WSD and LRFD methods, and subject to different environmental load-to-dead-load ratios, thereby delineating the load-to-capacity ratios of rigs designed using theses methods under these different enviromental conditions. The comparative results are significantly advantageous in the leg design of jack-up rigs, and determine that the jack-up rigs designed using the WSD and LRFD methods with UC values differ by approximately 31 % with respect to the API-RP code basis. It can be observed that the LRFD design method is more advantageous to structure optimization compared to the WSD method.*

Key Words : *Design Criteria, Working Stress Design (WSD), Load and Resistance Factored Design (LRFD), Jack-up Rig, Leg Structures*

요 약 : 잭업 드릴링 리그 (Jack-up drilling rigs)는 해양자원개발 분야 중 석유 및 가스 탐사 산업에서 널리 사용되는 대표적인 해양구조물이다. 이러한 잭업 구조물은 대체로 얕은 수심에서 사용하도록 설계되었지만 에너지 산업의 추세로 대수심 및 가혹한 환경 조건에서도 사용이 가능한 설계가 요구되고 있다. 이러한 잭업구조물의 운영환경 확장에 따라서 과도한 설계를 최소화하고 신뢰성 반영된 설계법이 요구되었다. 기존의 해양구조물 산업에서 잭업 구조물의 설계법은 사용(혹은 허용)응력 설계 (WSD: Working (or Allowable) Stress Design) 방법을 사용하여 설계가 되고 있었다. 이러한 설치환경변화에 따라서 충분한 신뢰성을 확보가 가능한 하중 및 저항계수 (LRFD: Load and Resistance Factored Design) 방법을 최근 개발되었고 규정화가 되었다. LRFD 방법은 통계적 기반으로 한 한계상태설계 개념으로 잭업구조물의 구성구조부재의 하중과 전산수치해석을 이용한 강도의 불확성을 하중 및 저항 계수로 표현하는 설계법이다. 개발된 LRFD 방법은 실제 잭업구조물 설계의 적합성 판단을 위하여 기존의 WSD 방법과의 정량적인 비교 분석이 반드시 필요하다. 따라서 본 연구는 기존의 WSD와 LRFD 방법으로 이용하여 실 잭업 구조물의 레그 구조를 대상으로 상용유한요소해석코드를 이용하여 정량적인 UC (Unity Check) 값을 기반으로 비교 분석하였다. 분석된 결과로 다양한 환경하중조건 하에서 LRFD 방법을 사용하여 잭업구조물의 레그(Leg) 설계에서 상당히 합리적인 UC 값을 가지고 기존 대표적인 WSD기법 중에 하나인 API-RP 코드 대비 약 31 % 차이가 분석되었다. 따라서 LRFD 설계 방법이 WSD 방법에 비해 구조 최적화 및 합리적인 설계에 더 유리하다는 것을 확인할 수 있었다.

핵심용어 : 설계 기준, 사용(허용)응력 설계, 하중 및 저항 계수 설계, 잭업 리그, 레그 구조

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1. Introduction

Jack-up rigs have been widely used in the offshore oil and gas exploration industry (hereafter referred to as 'the offshore industry'). Although these rig structures were originally designed for use in shallow water, trends in the energy industry have led to growing demand for their use in deeper water (>150 m) and in harsher environmental conditions, such as those in the North Sea (Tan and Lu, 2003).

Jack-up units have traditionally been designed using the working(or/and allowable) stress design (WSD) method, in which all the uncertainty in loads and material resistance is combined in a single safety factor. However, a more recent and entirely new specification approach, known as load and resistance factor design (LRFD), has also been developed (DNV-RP-C104, 2011).

In LRFD, the uncertainties in loads and material resistances are represented by separate load and resistance factors, which are typically more and less than 1.0, respectively.

The American Institute of Steel Construction (AISC) approved LRFD for use by the offshore industry in 1994. To effectively use the LRFD approach, the differences between WSD and LRFD-based offshore structural design processes must be understood. The fundamentals of LRFD have been disseminated via professional training to offshore design engineers, geotechnical engineers, engineering geologists and others responsible for design of topside structures. However, the application of LRFD to the design of offshore jack-ups requires further quantitative understanding of its differences to and advantages over WSD. Few previous studies have investigated this issue in theory and practice, and these are outlined as follows.

Williams et al. (1999) performed nonlinear FEM analyses using a two-dimensional model to investigate the dynamic response of an offshore jack-up unit. It was shown that the accurate non-linear modelling of both the legs and the spudcan footings had a significant effect on rig dynamics. Thus, this indicated that the widespread practice of modelling the footings as simple pinned supports may be too conservative for specific maritime conditions. Lewis et al. (2006) presented an approach that addressed some of the more important parameters for site assessment in the Gulf of Mexico using the guidelines from the Society of Naval Architects and Marine Engineers (SNAME) (2002). A comparative study of the structural reliability of a sample jacket and a sample jack-up rig (Morandi et al., 1999) yielded guidelines and results for the candidate methodologies, with an emphasis on jack-up rigs.

Tan et al. (2003) proposed an innovative method to minimise the occurrence of localised failure and collapse during installation. Numerical simulations were carried out under various loading and boundary conditions, yielding data that are a valuable resource when re-designing a jack-up rig structure and can be used as guidance for site installation.

In offshore rig structural design, design constraints are frequently referred to as limit states. In contrast to WSD, limit state design (LSD) explicitly considers limit states, which aim to define the various conditions under which the structure may cease to fulfill its intended function. For these conditions, the applicable load-carrying capacity is calculated and used in design or strength assessment as a limit for the related structural behaviour (Paik and Thayamballi, 2007).

LRFD and WSD loads are not directly comparable because they are treated differently by the design codes. That is, LRFD loads are generally compared with full-strength components or members, whereas WSD loads are compared with members or components having allowable values that are less than the full strength. To determine the comparative demands of these two design methodologies (i.e., to discern which methodology results in larger members), it is necessary to 'unfactor' the load combinations using the specific strength and allowable stress requirements of the material. Additionally, there are times when an engineer will know the capacity of a member relative to a limit state but will also need to know the actual loads.

In this context of evaluating applicable design methods for jack-up units, previous research is incomplete and has not fully described the distinctive features and benefits of using LRFD under environmental loading conditions. Therefore, in this paper we aim to provide a basis for understanding the differences between WSD and LRFD, and the benefits of LRFD for jack-up unit design.

2. Limit states design

Limit states are classified into four categories: serviceability limit states (SLS), ultimate limit states (ULS), fatigue limit states (FLS) and accidental limit states (ALS) (Paik and Thayamballi, 2007). SLS represents the exceedance of criteria governing normal functional or operational use. ULS represents the failure of the structure and/or its components, usually when subjected to the maximum or near-maximum values of actions or action effects. FLS represents damage accumulation (usually fatigue cracking) under repetitive actions, often considered on a component-by-component basis. ALS represents situations of accidental or abnormal events (Paik and Thayamballi, 2007). In the classification of LSD for this study, candidate limit states will be re-categorised as strength and serviceability limit states.

2.1 Strength limit states

Strength-based limit states are potential modes of structural failure. For steel members, the failure may be either yielding (permanent deformation) or rupture (actual fracture). The strength-based limit state can be written in the general form of Eq. (1):

$$\text{Required Strength} \leq \text{Nominal Strength} \tag{1}$$

where required strength is the internal force that engineers derive from numerical analysis of the structure being designed.

For example, when designing a beam, the required strength may be the maximum moment (M) computed for the beam. The nominal strength is the predicted capacity of the beam, such as the maximum moment (M_n) that the beam is capable of supporting (which is a function of the stress capacity of the material and the section properties of the member).

2.2 Serviceability limit states

Serviceability limit states are those conditions that are not strength-based but may still render the structure unsuitable for its intended use. The most common serviceability limit states in structural design are deflection, vibration, slenderness and clearance. Serviceability limit states can be written in the general form of Eq. (2):

$$\text{Actual Behavior} \leq \text{Allowable Behavior} \tag{2}$$

Serviceability limit states tend to be less rigid requirements than strength-based limit states because they do not concern the safety of the structure nor tend to put human life or property at risk.

In industrial practice, some engineers find it useful to divide the left-hand side of the limit-state inequalities by the right, such that the required strength divided by the nominal strength and/or the actual behaviour divided by the allowable behaviour is less than 1.0. The resulting simplified formula is useful for two reasons: it makes comparison easier (as the resulting unit check-value must be less than 1.0), and the resulting number simply indicates the percentage of capacity used. Determining this percentage helps engineers to decide which limit states are critical when optimising a complex design problem.

The relationship used in applying WSD and LRFD takes the following general forms (Eqs. (3) and (4)):

$$\frac{R_n}{FS} \geq Q_d + \gamma(Q_{t1} + Q_{t2}) \qquad \text{for WSD} \tag{3}$$

$$\Phi R_n \geq \gamma_d Q_d + \gamma_{t1} Q_{t1} + \gamma_{t2} Q_{t2} \qquad \text{for LRFD} \tag{4}$$

where R_n = nominal resistance; Q_d = nominal dead load effect; Q_{t1}, Q_{t2} = nominal transient load effects; γ = load combination factor; FS = Factor of Safety; γ_d, =load factor with nominal dead load effect; γ_{t1}, γ_{t2} = load factor associated with the i th load effect, Φ= resistance factor

In practical WSD, FS can range from approximately 1.2 to 6.0 (ASME, 2017), depending on factors such as the type of problem being evaluated, the model used to estimate resistance and the experience of the designer. LRFD represents a more rational approach by which the more significant uncertainties listed above (i.e., load and material resistance) can be quantitatively incorporated into the design process. The LRFD Specification (American Petroleum Institute [API], 1993) sets guidelines for how the basic LRFD equation or relationship is defined. Limit-state concepts are currently used in the LFD American Concrete Institute (ACI, 1995) design code and the LRFD AISC (1989) specifications for design of steel-based offshore structures. Several other countries have also adopted the limit-state design code format for design of offshore structures.

3. Applied preliminary design calculation

To provide a basis for understanding the differences between WSD and LRFD with respect to jack-up unit design, we conducted an industrial structural analysis and applied the design methodology of leg structures under in-place conditions. Simplified hull and detailed leg structures were simulated using commercial FE code (SACS, 2016), with all hull loads and environmental loads being included in the simulated computer model. The members and joints were checked for combined bending and axial loads, in accordance with the criteria of both components in the recommended rules and codes.

3.1 Target Jack-up rig

The objective of the global in-place analysis for the leg structure was to ensure that the unit was capable of safely supporting the intended lightship, deck facilities and payload in the operating and survival environmental conditions. Therefore, the selected target jack-up was based on a commonly designed jack-up drilling rig, which is an independent three-leg self-elevating unit with a cantilevered drilling facility, as shown in Fig. 1. Table 1 indicates the main dimensions, weights and environmental loads in storm survival conditions for the jack-up drilling rig, and the properties of the leg structures were as indicated in Table 2. The purpose of our preliminary overall basic design was to ensure the survivability of the unit under elevated risk design conditions. The overall strength of the leg and the overturning stability of the unit were therefore verified.

The general dimensions of the target unit were 88.8 m overall length, 105.1 m width and 12 m hull depth. The legs were of a three-chord open truss X-braced structure, 199 m long with spudcans of area 380 m^2. The jack-up unit consisted of a near-triangular pontoon-shaped hull, three open-truss legs, three sets of rack-and-pinion elevating systems and three sets of hull-to-leg fixation systems per leg. The jacking structures consisted of three double columns, connected to the hull around the leg well at the lower side by bracings at the top above the main deck. The jacking structures also included lower leg guides, supports for the fixation systems and jacking units, and upper leg guides supported at the tip of the rack, as shown in Fig. 1.

Effective member lengths were used to ensure compliance with the SNAME RP 5A-5 code (SNAME, 2002) as well as API guidance. Buoyancy was included in all elements below the wave crest. Accurate mass and added mass distributions were used to ensure the natural periods of the platform were appropriate. The hull sagging mode, owing to dead weight, was considered.

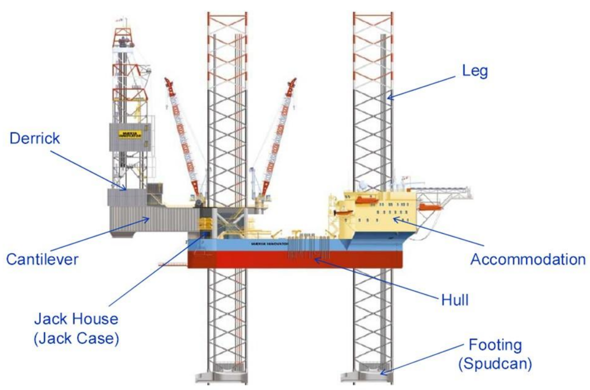


Fig. 1. A typical jack-up drilling unit (Ma et al., 2019).

Table 1. General model input

Description	Unit	Survival Condition
Limit State	-	ULS
Max. water depth	m	135.0
Max. wind speed	m/sec	45.0
Max. wave height	m	25.0

Table 2. Leg property

Type of legs	3-chord truss type
Chord distance	18.0 m
Type of chords	Split-pipe with opposed teeth rack
Thickness of rack in chord	210mm thick and 1050mm wide
Bracing type	Crossed fully “X” type bracing
Chord min. yield stress	690 MPa
Modulus of elasticity (E)	206 GPa
Shear Modulus (G)	80 GPa
Poisson’s ratio	0.3
Density	Steel = 7.85 tonne/m^3 Seawater = 1.025 tonne/m^3
Type of spudcans	Skirt type

3.2 Design procedures

The analysis of the structure was primarily based on the guidelines provided in SNAME RP 5A-5 (SNAME, 2002). The leg was designed to withstand the loadings resulting from the hull's 1-year and 50-year return environmental forces during its operating life. Details of the procedure are provided in Fig. 2.

In this procedure, all the members in the leg structure were modelled as beam elements. The hull was idealized as a grillage and equivalent properties of the hull were assigned. Members forming the leg-to-hull connection were modelled as axial compression-only elements with respective initial gaps based on a backlash calculation. Spudcans were modelled as rigid beams connecting the leg chords to the pinned support, which was at half the depth of spudcan penetration. Gravity loads such as the self-weight of legs, lightship weight and payload were applied to the structure. Lightship weight and payload were applied as uniform loads and global moments were applied to correct the centre of gravity (CoG) of the applied loads. The flow chart of the global in-place analysis for the leg structure is shown in Fig. 2.

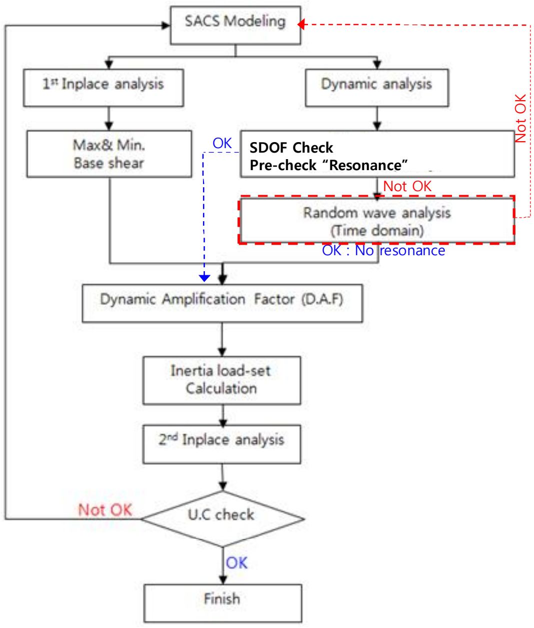


Fig. 2 Analysis flow-chart of leg.

The maximum and minimum values of base shear were obtained at the end of the spudcan during the first step of the static calculation, then the natural frequency of the jack-up rig was calculated using modal analysis. A dynamic amplification factor (DAF) can be easily obtained at the practical engineering stage by using the single degree of freedom (SDOF) method, as given by Eq. (5).

Although this method allows a rapid calculation of the DAF, resonance-stimulated excitation sometimes occurred in the region where the hull and wave periods were similar. After SDOF evaluation, structural changes and reinforcement have been examined, but this is a problem because it is difficult to accurately calculate the solution when using SDOF. In this case, random wave analysis based on the time domain was performed to calculate the correct DAF; when the DAF was determined, re-calculation was performed by updating the inertial load-set to the initial static analysis according to the heading angles. Finally, the code check was performed and iterative calculation was conducted until the allowable criteria were satisfied.

Basic environmental loads were applied to the structure in various combinations with the gravity loads. The structure was analysed for the applied loads, taking into account the P-△ effect (i.e., the second-order displacement), thus:

- 5 % damping is assumed (SNAME, 2002)
- The structure is also checked for the available FS against overturning.

3.3 Environmental consideration

3.3.1 Current Blockage Factor

The current blockage was taken into account by reduction of the far field current velocity, depending on the hydrodynamic drag coefficient (C_D) of the leg. The current blockage factor of 0.92 (Lewis and Brekke, 2006; Morandi et al., 1999) was used in this analysis.

3.3.2 Hydrodynamic Coefficients

The hydrodynamic properties of the leg were calculated through SNAME RP 5A-5 (SNAME, 2002). The C_D values were based on tests of both chords and complete legs. The chord racks and scales, diagonal bracings, span breakers and leg piping members were taken into account as structural and non-structural elements. Limited shielding of parts of the leg piping was taken into account, depending on wave direction. The hydrodynamic coefficients for drag and inertia, C_D and C_M, were based on the reference values of cylindrical bodies, as shown in Table 3.

Table 3. Hydrodynamic coefficient (ISO, 2016)

Part	C_D	C_M
Wind	0.50	-
Water (smooth)	0.65	2.0
Water (rough)	1.00	1.8

3.3.3 Wave Kinematic

The kinematic reduction factor of 0.86 was applied for the present design conditions using SNAME RP 5A-5 (SNAME, 2002). Second-order kinematics were also calculated for second-order wave surface elevation, while Stokes' fifth-order wave theory was used to calculate the period and height of the waves.

3.4 Dynamic Amplification Factor (DAF)

The overall design calculations were based on a quasi-static approach, which did not directly account for the dynamic response of the unit. However, the displacement of the hull mass due to the dynamic wave loads amplified the extreme reactions in the legs. This dynamic response effect was thus included in the our calculations, resulting in a DAF and inertial loads. The DAF was used to calculate the inertial load factor, which was multiplied by the wave- and current-loading to determine the inertial load.

$$DAF = BS_{dynamic} \ / \ BS_{quasi \ static} \tag{5}$$

where $BS_{dynamic}$ is maximum dynamic wave/current base shear, BS_{quasi} is maximum quasi-static wave/current base shear

The DAF can be calculated using the multi-degree of freedom (MDOF) model (Eq. (5)), with random excitation based on a 3-hour time domain simulation. This simulation was performed using the data shown in Table 4. The calculated DAF was used to estimate an inertial load-set, which represented the contribution of dynamics. The inertial load-set was calculated using the following formula (Eq. (6)):

$$F_{in} = (DAF-1)\frac{[BS_{(Q-S)Max.} - BS_{(Q-S)Min.}]}{2} \tag{6}$$

where $Bs_{(Q\text{-}S)max}$ is maximum quasi-static wave/current base shear and $Bs_{(Q\text{-}S)min}$ is minimum quasi-static wave/current base shear.

Table 4. Time domain simulation data

Items	Description
Wave simulation validity	Correct mean wave elevation Standard deviation = (Hs / 4) ± 1 % -0.03 < skewness < 0.03, 2.9 < kurtosis < 3.1
Time-step	< Tz/20, < Tn/20
Simulation time	60 minutes
Wave spectrum	JONSWAP
Damping	5 % of critical damping

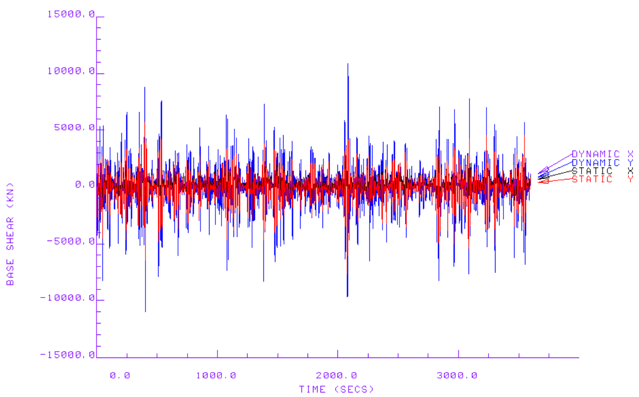


Fig. 3 DAF calculation under nonlinear dynamic analysis

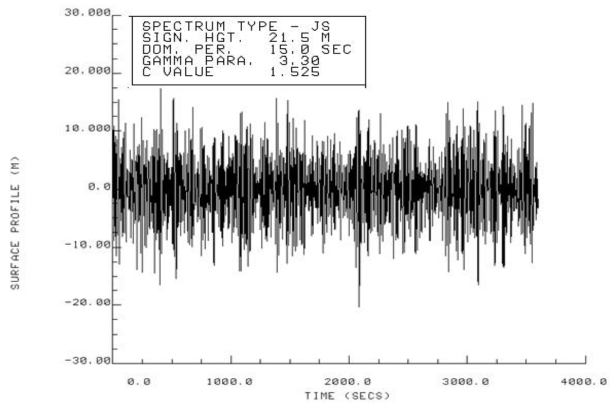


Fig. 4. Surface wave profile according to time, based on the JONSWAP wave spectrum.

The inertial load-set thus calculated was applied at the global CoG. The dynamic amplification was calculated as the ratio of the absolute values of the dynamic maximum and the quasi-static maximum, as shown in Fig. 3. Second-order wave kinematics were used in the calculation of the DAF. The JONSWAP wave spectrum was applied to the calculations, and indicated a 21.5 m wave height with a domain wave period of 15 second. The JONSWAP spectrum represents the conditions in the North Sea, and the density of maximum energy is comparatively narrow, so it is often used for conservative designs. The total wave surface profile is shown in Fig. 4.

3.5 Stability against Overturning Moment

The design requirement with respect to overturning is expressed by Eq. (7):

$$\gamma_s \le \frac{M_s}{M_o} \tag{7}$$

where M_s is overturning moment caused by environmental loads, M_o is stability moment caused by functional loads, and γ_s is safety coefficient, with a required value of 1.1.

The design requirement should ensure survival under the most unfavourable direction and combination of loads, and thus it is normally assumed that wind, waves and current are coincident in direction in such a scenario. The check-sheet for stability against overturning moment (OTM) is shown in Fig. 5. The results show that the our design has a sufficient safety ratio, as the stability against OTM exceeds the allowable criterion (10 % margin) at all heading angles.

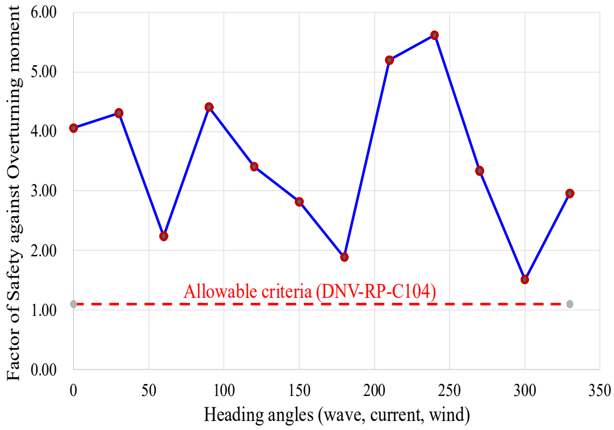


Fig. 5. Performance of safety factor with respect to allowable criterion as a function of heading angle.

Table 5 show the main results used to calculate the FS for the OTM of the jack-up rig. First, Mx and My were calculated by static analysis under various heading angles, and the OTM was calculated using the root mean squares of the two values. Each moment arm generates an OTM, which was calculated according to the heading angle. Then, the stabilising moment was calculated, and the ratio between moments was found using Eq. (7).

Table 5. Safety Factor of overturning moment as a function of heading angles from 0 to 330 degrees

Deg.	Mx (kN-m)	My (kN-m)	OTM (kN-m)	Stabilizing Moment (kN-m)	RT
0	33	3,448,522	3,450,000	14,200,000	4.12
30	-1,376,423	2,457,199	2,820,000	12,200,000	4.33
60	-2,611,557	1,498,554	3,010,000	6,630,000	2.20
90	-2,545,253	-1,032,471	2,750,000	12,200,000	4.44
120	-3,529,465	-2,022,550	4,070,000	14,000,000	3.44
150	-2,105,047	-3,698,218	4,260,000	11,900,000	2.79
180	-145	-3,472,281	3,470,000	6,530,000	1.88
210	1,623,274	-1,724,453	2,370,000	12,300,000	5.19
240	1,816,901	-1,781,892	2,540,000	14,300,000	5.63
270	3,589,961	8,196	3,590,000	12,100,000	3.37
300	3,717,248	2,136,838	4,260,000	6,370,000	1.50
330	1,978,202	3,499,934	4,020,000	12,000,000	2.99

Note: RT is the ratio of the stabilising moment to the OTM value, with a maximum allowable value of 1.1, and Mx and My are the moments in the x and y directions.

3.6 Summary of analysis assumption and set-up

Various assumptions were made involving the structural analysis and design, based on previous design data and the environmental conditions of the target jack-up unit.

Members were assumed to be coincidental at work points. Brace offsets were modelled as a node where the brace offset value was more than 25 % of the diameter of the pipe. Deterministic (regular) waves were used to calculate the hydrodynamic loads on the structure. A kinematic reduction factor of 0.86 was used to account for the conservatism involved in the deterministic approach. The in-place analysis was carried out for a range of wave periods, as per SNAME RP 5A-5 and API requirements. The fixity level was assumed to be at 1.5 m, i.e., half the depth of the spudcan penetration. The current blockage factor was taken as 0.92 for all headings. Hydrodynamic loads on individual members were calculated using Morison's equation. No shielding or interaction effects within the structure were considered. The wave force on non-tubular and/or complex geometries was calculated using an equivalent diameter, which was in turn was based on the circumscribing circle.

The basic drag and inertia coefficients for submerged members were increased by 5 % to account for wave forces on the anode and other miscellaneous members. Modal analysis of the structure was carried out with the above-mentioned loads to generate the first and second natural frequencies of the structure. DAFs and inertial load-sets were calculated using an MDOF model based on a nonlinear dynamic simulation with the time domain approach.

4. Results

Tables 6-7 compare the basic FS, load and resistance factors of LRFD and WSD. The components are divided into two categories, namely tubular and non-tubular members. The former are subject to the API's RP criterion and the latter to the AISC criterion. The FS values of the WSD design are less than those of the LRFD design, as shown in Table 6. This implies that WSD is more conservative than LRFD as a design method. However, the opposite is true for the loading values, with those for WSD greater than those for LRFD.

Table 6. Safety Factor of LRFD and WSD

Type	Components	Safety Factor		
		LRFD		WSD
Tubular	Codes	API	SNAME	API
	Axial tension	0.95	0.90	0.60
	Compression	0.85	0.85	-
	Bending	0.95	0.90	0.75
	Shear	0.95	0.95	0.40
	Hoop	0.80	0.80	0.67
Non-Tubular	Codes	AISC	SNAME	AISC
	Axial tension	0.90	0.95	0.60
	Compression	0.90	0.85	0.60
	Bending	0.90	0.95	0.60
	Shear	0.90	0.95	0.60

Table 7. Load Factor of LRFD and WSD

Components	Load Factor		
	LRFD		WSD
Codes	API, AISC	SNAME RP	API, AISC
Dead load	1.10	1.00	1.00
Wind	1.35	1.15	1.00
Wave	1.35	1.15	1.00
Current	1.35	1.15	1.00
Material	1.15	1.0	1.00

Therefore, it is difficult to decide which methodology is more conservative. To resolve this problem, a series of dynamic analyses were performed by varying two main parameters, namely wave height and total elevated weight. All the steel members were circular XS pipes with an outer diameter of 0.6 m and a maximum thickness of 0.12 m, made of ASTM Grade B steel to withstand cold weather conditions. The minimum yield stress for this steel is 690 MPa.

As shown in Fig. 6, in the WSD case, the maximum combined stress and axial stress are both within the yield-stress limit, having unity check (UC) values of 0.97 and 0.93, respectively, where UC was calculated as the ratio between the maximum combined stress and the allowable stress. All the structural members withstood the applied environmental loading, having adequate FS with respect to the failure modes of the ASTM Grade B steel pipe used for the structure. The critical loading condition took place at approximately 300 degrees under maximum base shear, owing to a large increase of the overturning moment.

In the LRFD case, the maximum combined stress and axial stress were again both within the yield-stress limit, giving UC values of 0.75 and 0.72, as shown in Fig. 7. The maximum UC value of LRFD according to the SNAME RP 5A-5 criteria was dramatically reduced raby approximately 30 % a compared with the WSD results. This was attributable to both the load and resistance FS values. The LRFD also gave lower UC values in the case of API criteria, as shown in Fig. 8.

The maximum combined UC values for brace members are shown in Fig. 9. The differences between WSD and LRFD are almost 30 %, as they were for non-tubular members. However, the maximum UC is still substantially greater than the allowable value.

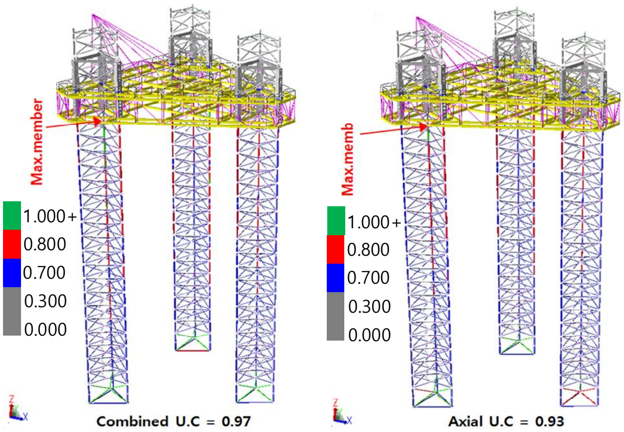


Fig. 6. Unit check results (Max.wave height = 27 m, WSD).

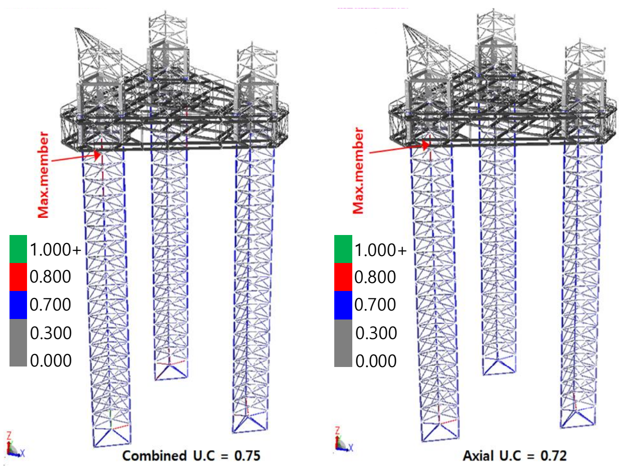


Fig. 7. Unit check results (Max.wave height = 27 m, LRFD).

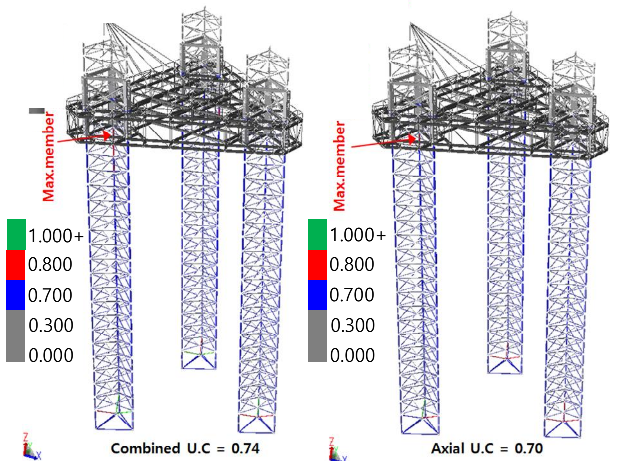


Fig. 8. Unit check results (Max.wave height = 27 m, API-LRFD).

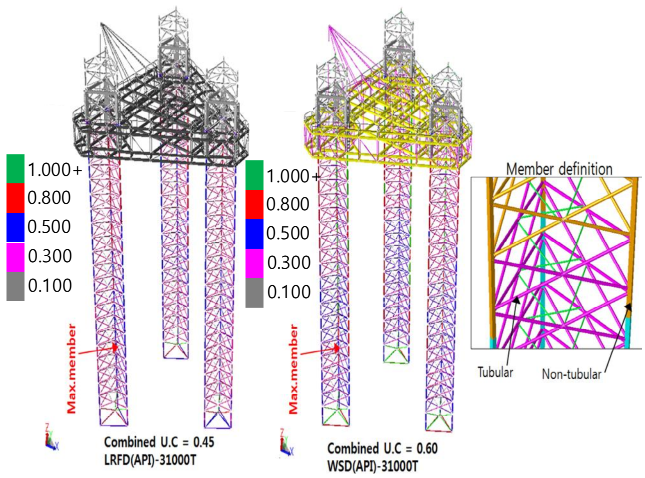


Fig. 9. Comparative U.C results for brace member of WSD and LRFD (Max.wave height = 25 m, weight = 31,000 ton).

Table 8. Comparative U.C results varying environmental load

Methods	LRFD				WSD	
	API, AISC		SNAME RP 5A-5		API, AISC	
W.H(m)	T.B	N.T.B	T.B	N.T.B	T.B	N.T.B
25	0.42	0.69	0.38	0.71	0.56	0.92
27	0.46	0.74	0.42	0.75	0.62	0.97
29	0.51	0.78	0.46	0.80	0.69	1.03

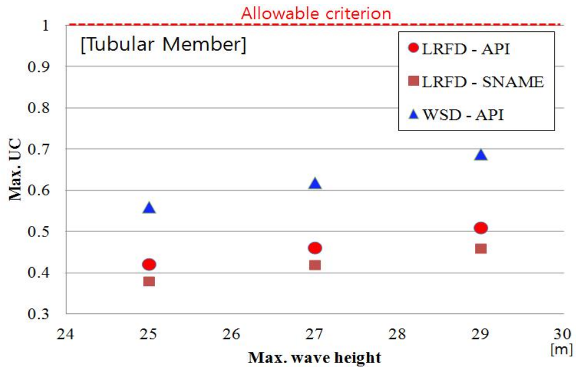
Notes: W.H=Wave Height, T.B=Tubular, N.T.B= Non-Tubular, U.C= Unity Check

Table 9. Comparative U.C results varying elevated weight

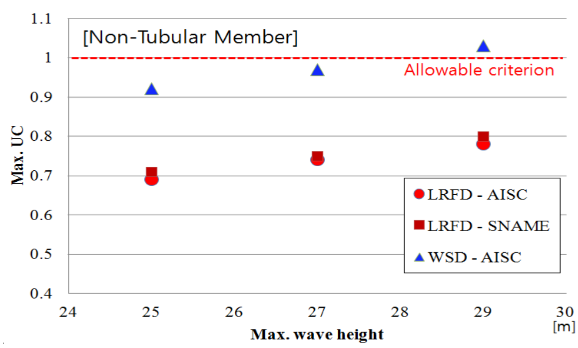
Methods	LRFD				WSD	
	API, AISC		SNAME RP 5A-5		API, AISC	
E.W(ton)	T.B	N.T.B	T.B	N.T.B	T.B	N.T.B
27,180	0.42	0.69	0.38	0.71	0.56	0.92
29,000	0.43	0.72	0.39	0.74	0.58	0.95
31,000	0.45	0.75	0.41	0.77	0.60	1.00

Note: E.W= Elevated Weight, T.B= Tubular, N.T.B= Non-Tubular, U.C= Unity Check

A summary of the UC‘s under various environmental conditions and elevated weights is shown in Tables 8, 9 and Figs 10, 11. The WSD overestimated the UC values by approximately 30 % compared with the LRFD method. This implies that industrial designers should be careful, to consider the actual characteristics of the problem at hand when selecting a design method for offshore structures, such as project type, loading condition, required specification and owner requirements.

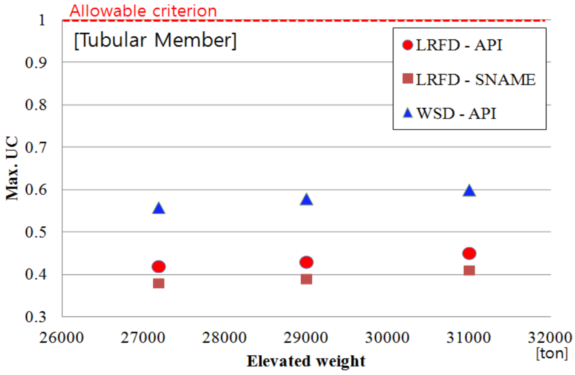


(a) Tubular member

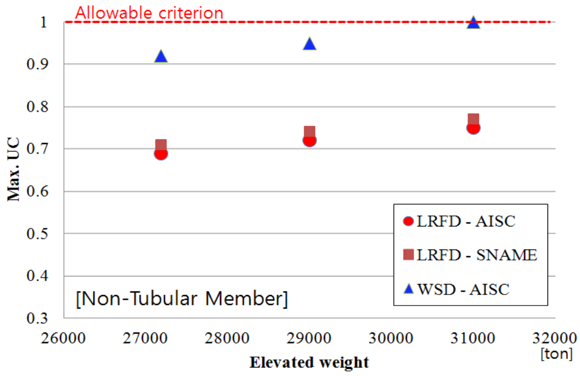


(b) Non-Tubular member

Fig. 10. Unit check results varying environmental condition.



(a) Tubular member



(b) Non-Tubular member

Fig. 11. Unit check results varying total elevated weight.

5. Conclusion and remarks

A series of nonlinear random wave analyses were performed to investigate the DAF of the relationships between static and dynamic base shear components. A static analysis with reflected on the P-△ effect was used to evaluate the response under varying environmental loads and total elevated weights. Specifically, lattice leg structures designed using WSD and LRFD were modelled at different environmental load-to-dead-load ratios, to investigate the differences between two design methods (WSD and LRFD) by comparing the load-to-capacity ratios of the jack-up rigs designed by each method under these conditions. The following conclusions can be drawn:

LRFD can account for the variability in both resistance and load, but its utility in the design stage is limited because the resistance factors are not constant.

The comparative results in this paper will be very helpful for leg design of jack-up rigs, and show that the WSD and LRFD give UC values that differ by approximately 31 % according to the API-RP code basis. It can be seen that the LRFD design method is more advantageous to the structure optimization compared to the WSD.

The critical loading condition occurs at approximately 300 under maximum base shear, owing to the large increase of the overturning moment induced by environmental loads such as wave and wind.

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