THE DERRICK (*ONSHORE*) STRUCTURE CAPACITY ANALYSIS IN WIND LOAD COMBINATION EFFECT BASED ON SNI 1729 2015

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ABSTRACT

Steel truss tower structures for onshore oil drilling infrastructure (derricks) had been located in oil-producing areas, Tarakan was one of them. Several tower structures as Petroleum Batavische Maschapiij heritage had been found in incomplete structure such in bracing elements lost. This research analyzed the incomplete tower structure capacity were based on SNI 1729-2015. The service load design (ASD) and the limit load design (LRFD) had been conducted in this research. The loading combination had accorded into SNI 1727 2013 with a research focused on the wind loads combination. The tower structure modeling used 3D truss element idealization by utilizing numerical software were based on the finite element method, SAP2000. The tower structure model had been classified into 2 consecutives model, named: the complete tower model (CD) and the incomplete tower model (ICD). The incomplete tower model was consisted of 3 representatives of the existing tower structure prototype in Tarakan City, respectively the Kampung Satu tower structure (ICM-K1), the Kampung 4 tower structure (ICM-K4), and the Kampung 6 tower structure (ICM-K6). The three incomplete derrick models had been distinguished by the number of bracing elements lost. The results showed that the capacity of the incomplete tower structure (ICM-K1, ICM-K4, and ICM-K6) had strong enough to accept a combination of wind loads. The service load design (ASD) had the conservative member capacity that the limit design (LRFD). The tension member capacity decreased due to number of bracing elements lost had more significant than the decreasing of the compression member capacity. The ICM-K6 model with the largest number of bracing elements lost had the lowest tension and compression member capacities.

Keywords: *analysis*, *onshore*, *tower*, *truss, wind*

INTRODUCTION

The steel truss tower structure as one of the infrastructure for onshore oil drilling exploration. This structure had also known as a derrick structure where located in oil drilling areas in Indonesia, the city of Tarakan was one of them. Several derrick structures had been found in incompleted structure. This was caused by the age of the structure was very old and considered that some of them were the legacy of *Batavische Petroleum Maschapiij*, an oil exploration company of the Dutch-East Indies colonial government. Some of the bracing elements of the derrick structure had been lost.

The steel building design standards (codes) of the derrick structure had not easy to be traced. The steel material type that was used in the derrick structure was difficult to be identified since almost of the steel member surface had been covered by uniform corrosion and pitting corrosion. API RP 4G (2019) had provided and recommended the inspection procedures, maintenance and repair of the drilling item as well as derrick structures. API had also developed and documented a new methodology for determining wind loads on drilling structures, such as piles and derrick structure.

Miftahul and Azis (2020) had conducted the research on the derrick structures performance evaluation by using three consecutive steel design codes: AISC-ASD 1983; AISC-LRFD 1993; and AISC-2010. The results showed that the derrick structure had satisfied the stability and strength requirements for every codes. Jian, et al. (2015) had conducted the research on static and dynamic analysis of the derrick structure. The results showed that one of the derrick structures had vibrated when operating with a rotary table rotational speed of about 120 rpm in a frequency of 2.0 Hz.

SNI 1727 2013 had provided the load combinations that were carried on the structure. The load combinations had been classified for 2 consecutives design criteria: the service load (Allowable Strength Design, ASD) and the limit load (Load Resistance Factor Design, LRFD). SNI 1729 2015 had provided the steel member capacity such in tension and compression member design (Equation (1)).

$$P_n \geq P_u \text{ dan } P_n \geq P_a \tag{1}$$

where

P_n : nominal member capacity, N

P_a : actual load, N

P_u : factored load, N

The tension member capacity design had been determined in Equation (2).

$$P_n = F_y.A_g \tag{2a}$$

$$P_n = F_u.A_{eff} \tag{2b}$$

where:

F_y : steel yielding stress, MPa

F_u : steel ultimate stress, MPa

A_g : Gross area section, mm^2

A_{eff} : The effective cross section, mm^2

The Equation (2b) had satisfied for the tension member in fracture design criterion. The design had the bolt holes presence consideration. The steel yield stress (F_y) and the ultimate stress (F_u) had been determined accorded into the steel type that was used in the structure.

SNI 1729 2015 also had provided the compression member capacity design (Equation (3)).

$$P_n = F_{cr}.A_g \tag{3}$$

Where F_{cr} was the steel critical stress (MPa). The number of F_{cr} had been determined by the slenderness ratio (KL/r) of the compression member.

Guan, et al. (2014) had conducted the research on offshore module drilling rigs used load capacity assessment analysis. The finite element method had been combined with the field test method, had evaluated the stress concentration in the main derrick member. Solazzi (2017) had also carried out the research on the large derrick structures design (the length of the main boom was 80 m and the payload was 60 t) took into account the dynamic effects of load transfer. The results showed that the dynamic action caused by the sudden load release had much higher than that induced by using the crane under normal load conditions and the magnitude of this action was correlated with the geometrical configuration of the derrick structure.

SNI 1727-2013 had also provided the wind load design were based on the wind velocity design.

$$F_w = q_z.G.C_f.A_f \tag{4}$$

where:

$$q_z = 0,613.k_z.V^2 \qquad (5)$$

q_z = velocity pressure

G = gust factor

V = wind velocity (kph)

A_f = open construction area (m^2)

C_f = wind load coefficient

This research aims to analyze the strength capacity of the existing derrick structure in Tarakan City by considering the effect of bracing element decreased.

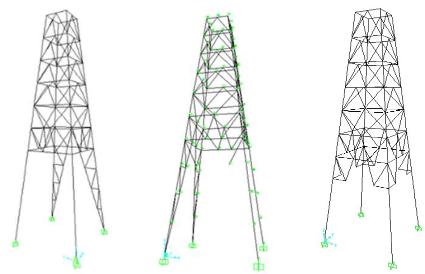
RESEARCH METHOD

This research modeled the steel truss tower (derrick) structure used the 3D truss elements idealization by utilizing SAP2000. The prototype of the derrick structure respectively where located in Kampung 1 (Figure 1(i)), Kampung 4 (Figure 1(ii)), and Kampung 6 (Figure 1((iii)) Tarakan city.

(i) (ii) (iii)

Figure 1. Derrick structure protype

The derrick structure modeling in this case had been divided into two models, namely: the complete derrick structure model (CDM) and incomplete derrick structure model (IDM). The IDM model had been distinguished by the number of bracing elements lost accorded in its location respectively, IDM-K1, IDM-K2, and IDM-K3. The number of bracing elements lost had been determined based on the existing derrick structure prototype where located. The 3D truss incomplete derrick structure model had been presented in Figure 2.



(i) (ii) (iii)

Figure 2. The 3D truss IDM

The static analysis had been performed in this research. The load combination for the derrick structure in this study had been determined below:

The service load

$D + 0,6W$

$0,6D + 0,6W$

The limit load

$1,2D + 1,0L + 1,0W$

$0,9D + 1,0W$

where D was permanent load, L was live load and W was wind load. the supplementary load had been neglected in this case because the existing derrick prototype had no longer operating.

This research emphasized on the wind load combination that was carried on the derrick structure system. The wind load had been obtained from the calculation of the design wind velocity. The log-Pearson Type III had been used statistically to estimate the wind velocity design with a return period of 100 years. The velocity data were based on the Tarakan Meteorology, Climatology and Geophysics Agency (BMKG) as shown in Table 1. The observation locations were in 3.3 North latitude and 117.5 East longitude with an elevation of 6.00 m.

Table 1. The maximum wind velocity

Month	Wind velocity (knots)					
	2009	2010	2011	2016	2017	2018
Jan	9	6	8	6	5	9
Feb	11	7	10	6	9	6
Mar	9	7	6	6	7	5
Apr	12	5	7	5	6	6
May	8	5	10	10	6	4
Jun	15	6	8	9	9	8
Jul	8	6	12	9	7	8
Aug	6	9	10	13	9	6
Sept	11	9	11	10	8	6
Oct	9	13	6	7	15	6
Nov	9	4	9	6	7	6
Dec	6	11	9	6	8	6
Total	113	88	106	93	96	76
Max	93	96	76	13	15	9
Mean	9	7	9	8	8	6

Source: BMKG Tarakan, 2018

The wind load had modeled in a concentrated load that was carried on each node (joint) on derrick structure perpendicularly. The maximum wind load number had been assigned to the top node of the derrick structure then gradually decreased due to the nodal elevation decreased.

The wind load distribution had been assumed was occurred at one side of the derrick linearly. The triangle load distribution had a carried on in this case.

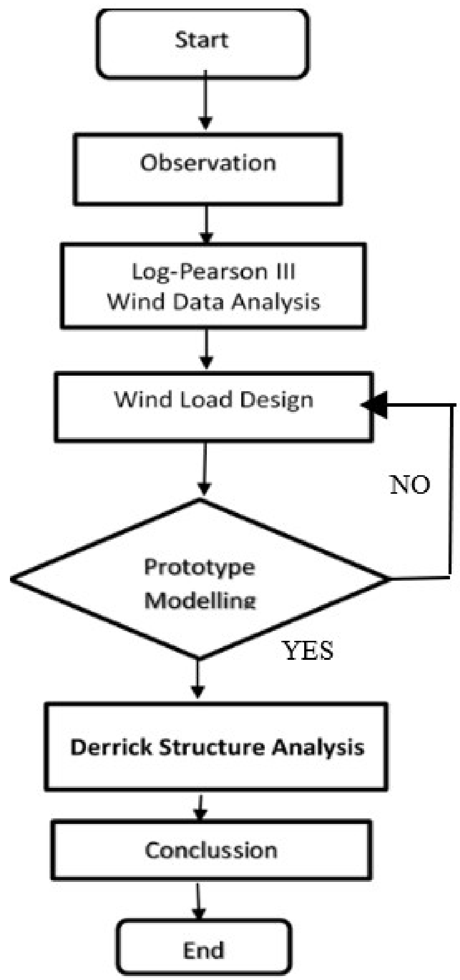


Figure 3. Flow chart

Figure 3 showed the research flow chart. In general, the research procedure was consisted in two steps, the member capacity analysis (tension and compression) and the evaluation of the member capacity decreasing due to the member stiffener (bracing) lost.

RESULTS AND DISCUSSION

The results of this study had been consisted of the wind load analysis results, the member capacity analysis results, and the evaluation of the member capacity decreased due to bracing element lost.

Wind Load Analysis

The statistical parametric of the wind velocity data had been presented in Table 2.

Table 2. Statistical parameter

	Remarks	Number
n	Data Num.	10
S_{xi}	Wind Vel.	79
X_r	Mean	7,9
$\sum(X_i - X_r)^2$		36,9
$\sum(X_i - X_r)^3$		-1,92
$\sum(X_i - X_r)^4$		226,02
S	Std. dev.	2,03
C_v	Var. coef.	0,26
C_v	Asym. coef.	-0,03
C_k	Curts. Coef.	0,27

Several statistical parameters had been evaluated such as standard deviation (S = 2.0 knots), coefficient of variation (Cv = 0.2), asymmetry coefficient (Cs = 0.01) and curtosis coefficient (C_k = 0.2). The selection of frequency type had been presented in Table 3.

Table 3. Frequency distributions

Type	Parameter	Number
Normal	$C_s = 0$	-0,032
Log Normal	$C_s/C_v = 3$, Cs Negatif	-0,125
Gumbel	$C_s = 1{,}1396$	-0,032
Log-Perason Type III	$C_k = 5{,}4$	0,267

Based on Table 3, the frequency distribution chosen for the wind velocity data distribution was the log-Pearson distribution Type III. Furthermore, the calculation of the design wind velocity design can be seen in Table 4.

Table 4. Wind velocity design (knots)

Month	Wind velocity (knots)					
	2020	2023	2028	2043	2068	2118
Jan	8	10	11	12	12	13
Feb	8	9	10	11	12	13
Mar	7	8	8	9	9	10
Apr	7	9	10	12	13	13
May	7	9	10	11	12	13
Jun	10	13	15	17	19	20
Jul	9	10	12	13	14	14
Aug	9	11	12	13	14	15
Sept	10	16	19	24	27	30
Oct	10	16	19	24	27	30
Nov	7	9	10	11	12	12
Dec	9	12	14	16	17	18

Based on Table 4, the maximum wind velocity design had been defined in 15 knots (27,78 kph). The wind load had been calculated used Equation (4) and had determined 382.84 kg (3828.40 N).

Member Capacity Analysis (2D case)

The derrick structure capacity analysis consisted of tension and compression member capacity analysis.

Table 5. Tension member capacity

ASD, P_a = 42178 N	LRFD, P_u = 70839 N
Yield criteria:	*Yield criteria*:
Ω_t = 1,67	$Ø_t$ = 0,90
P_n/Ω_t = 222467 N	$Ø_tP_n$ = 334368 N
Fracture criteria:	*Fracture criteria:*
Ω_t = 2,00	$Ø_t$ = 0,75
P_n/Ω_t = 163392 N	$Ø_tP_n$ = 245088 N
-OK_	-OK

The capacity analysis used the allowable strength design (ASD) and load resistance factor design (LRFD). The results of the tension member capacity analysis had been presented in Table 5.

Table 5 also showed that the steel profile that used (L100x100x10) as a tension member had satisfied the tension member design requirement. The two design criteria, respectively, Allowable Strength Design (ASD) and Load Resistance Factor Design (LRFD) produced a nominal tension load as the tension capacity (P_n) was smaller than the external load.

The compression member capacity analysis had also been carried out in this research. The slenderness ratio (KL/r) indicated that the compression member was included inelastic column criteria design. The results of the compression member capacity had been shown in Table 6.

Table 6. Compression member capacity

DKI, P_a = 51707 N	DFBK, P_u = 89991 N
KL/r: 56,97 Ω_t = 1,67 P_n/Ω_t = 178177 N -OK-	KL/r: 56,97 $\varnothing_t$ = 0,90 $\varnothing_t P_n$ = 267801 N -OK-

Table 6 also showed that the L100x100x10 BJ37 steel had satisfied for the compression member design requirement. The design criteria, respectively, Allowable Strength Design (ASD) and Load Resistance Factor Design (LRFD) had nominal compression load (P_n) larger than the external load.

Figure 4 showed the normal (axial diagram of the derrick structure in 2D-model. The normal force diagram shown in Figure 4 was a wind load combination (0,6D+0,6W) in ASD. Figure 4 also showed the deformation of the derrick structure due to the combination of wind loads. The displacement at one of the nodes had been also evaluated (14,39 mm). The nodal 35 (top) had been evaluated for the displacement in-x direction.

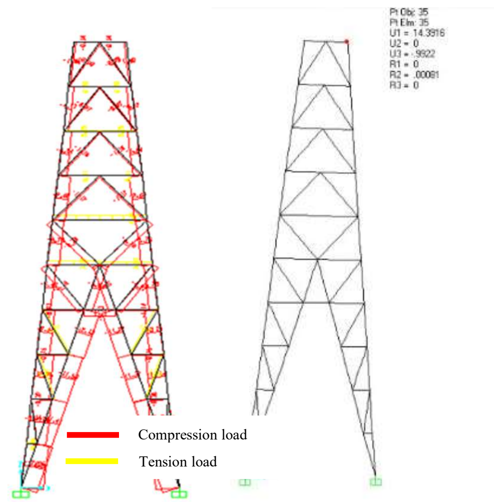


Figure 4. Normal force diagram (2D-case)

Member Capacity Decreasing (3D case)

The results of this study had also evaluated the decreasing of member capacity (tension and compression) due to the bracing element reduced.

Table 7. Derrick capacity decreased (ASD)

Model	Member capacity (%)	
	Tension	Compression
CDM-00	-	-
IDM-K1	18,91	10,76
IDM-K4	19,69	9,74
IDM-K6	20,64	8,44

Table 7 showed the percentage of member capacity decreased of the incomplete derrick model (K1, K4, and K6). The percentage od the bracing element lost had been evaluated. Figure 6 showed the comparison of the percentage derrick capacity decreacing due to the percentage bracing element lost curve.

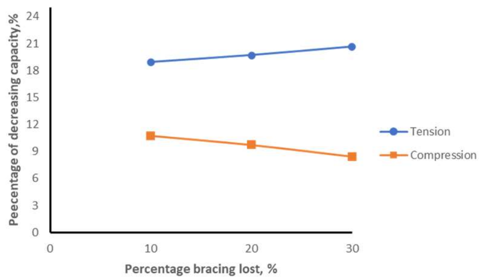


Figure 6. The decreasing derrick capacity

Figure 6 showed the percentage the percentage of member capacity decreased of the incomplete derrick model (K1, K4, and K6). The percentage of the bracing element lost had been evaluated. Figure 6 showed the comparison of the percentage derrick capacity decreacing due to the percentage bracing element lost curve.

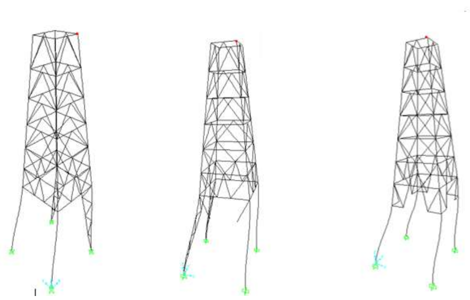


Figure 7. Deformed shape of IDM

Figure 7 showed the deformed shape of the incomplete derrick model correspondently: IDM-K1, IDM-K4, and IDM-K6. The deformed shape had been caused by wind load combination (0,9D +1,0W). Figure 7 also showed that the legs element of the derrick had supported the lateral load action caused by wind load. That was why the compression member had significant contribution on stability and strength of the derrick structure.

CONCLUSION

Based on the results of numerical modeling of derricks structure analysis, it can be concluded below:

1. Generally, the tension and compression members of the derrick structural system capacity satisfied the member design requirement for the wind load combination ($P_n \geq P_a$, ASD; $P_n \geq P_u$, LRFD).
2. The wind load combination had significant effect on the incomplete derrick structure, especially in the case bracing element lost (20,64% for tension and 18,93% for the compression)
3. The compression member had larger contribution to the strength capacity of the derrick structure then the tension member (30,70%).
4. This was important to provide a safety element for the IDM-K1 and IDM-K6 derrick structure prototypes considering the age factor of the derrick construction.

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REFERENCES

American Petroleum Institute. (2019). “API RP 4G 2019 Operation, Inspection, Maintenance, and Repair of Drilling and Servicing Structures”. API Recommended Practise (API). Washington DC.

Badan Standarisasi Nasional. (2015). “SNI 1729 2015 Spesifikasi untuk Bangunan Gedung baja Struktural”. Badan Standarisasi Nasional (BSN). Jakarta.

Badan Standarisasi Nasional. (2013). “SNI 1727 2013 Beban Minimum untuk

Perancangan Bangunan Gedung dan Struktur Lain”. Badan Standarisasi Nasional (BSN). Jakarta.

Guan, F., Zhou, C., Wei, S., Wu, W., dan Yi, X. (2014). “Load-Carrying Capacity Analysis on Derrick of Offshore Module Drilling Rig”. Petroleum Engineering Journal. 07. pp. 29-40.

Jian, H., Sizhu, Z., Lei, H., dan Xinmei, Y. (2015). “Structural dynamics modification for derrick of deep well drilling rig based on experimental modal test and frequency sensitivity analysis”. Journal Advances in Mechanical engineering. 07. Issue 09. pp. 1-7.

Miftahul, I. dan Azis, S. (2020). “Evaluasi Kinerja Struktur Menara Rangka Baja (*Derrick*) Berdasarkan Periode AISC”. Jurnal KACAPURI. 03. Issue 01. pp. 96-107.

Solazzi, L. (2017). “Design of a High Capacity Derrick Crane Considering The Effects Induced by Load Application and Release”. Journal of Applied Engineering Science. 15. Issue 409. pp. 15-24.