離岸風機支撐結構設計概述

中興工程顧問股份有限公司 電力及能源工程部
徐偉朝 博士/結構技師

2018年4月26日

離岸風機支撐結構設計概述

- 離岸結構概觀
- 離岸結構設計考量
- 耐震設計議題

離岸風機支撐結構設計概述

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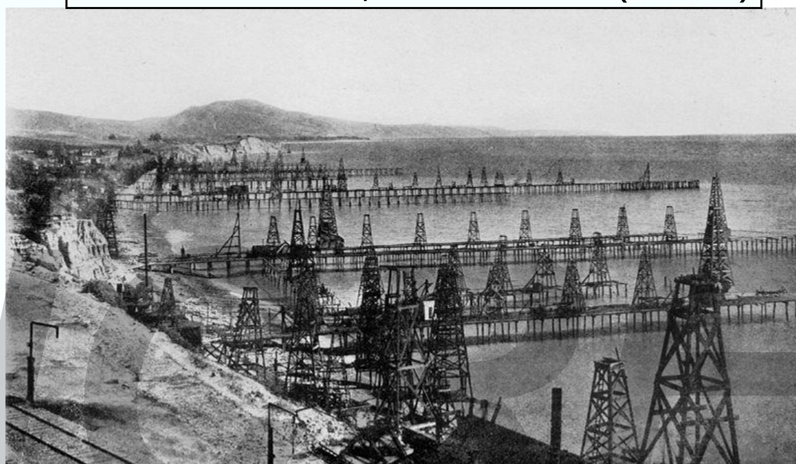
離岸結構發展

第一個離岸結構
Summerland, California (1896)



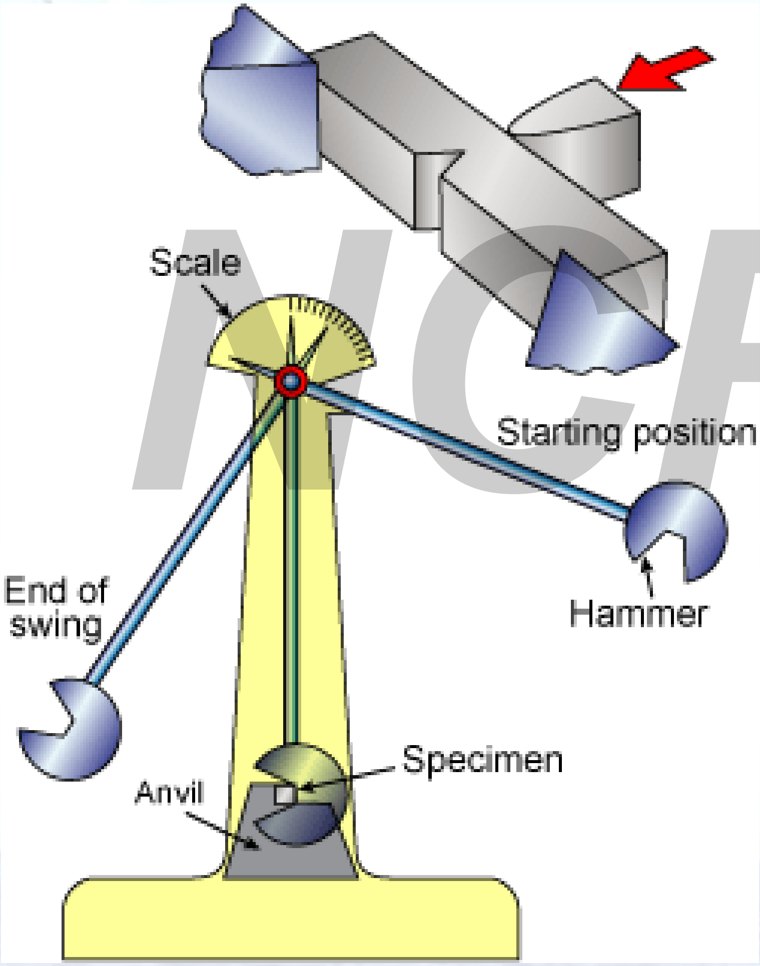
離岸鑽油平台結構
1937年始建於美國

離岸風力發電結構
1991年始建於丹麥

陸域與離岸結構之設計差異-疲勞

- 陸上: 車輛
- 海上:風、波、流、機械

陸域與離岸結構之設計差異-韌性

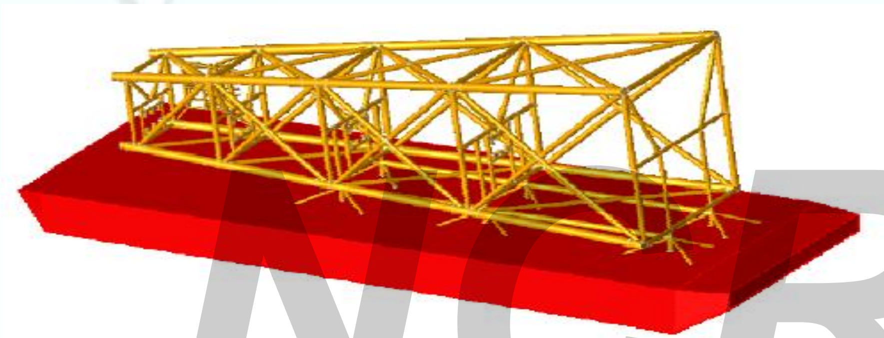


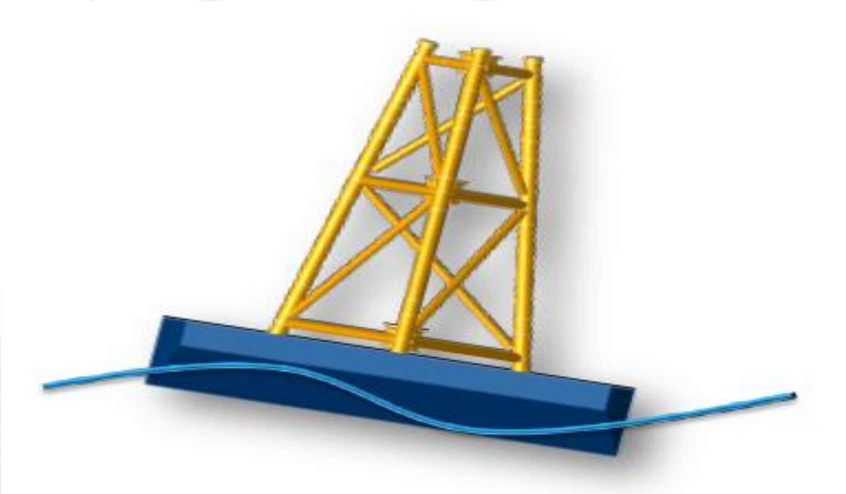
陸域與離岸結構之設計差異-腐蝕

陸域與離岸結構之設計差異-施工



陸域與離岸結構之設計差異-運輸與施工分析





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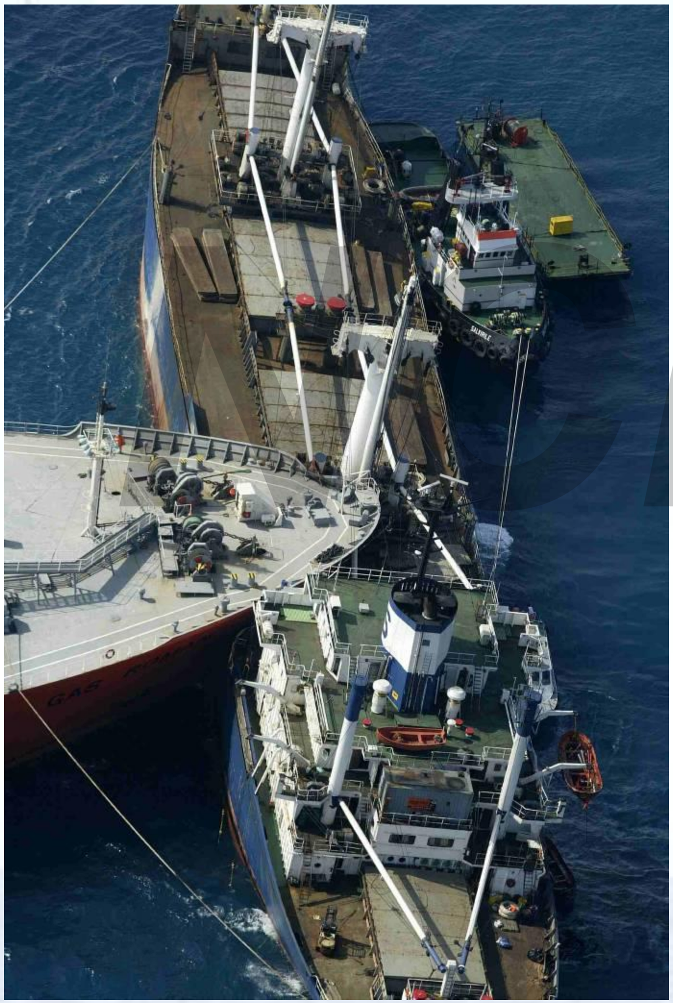
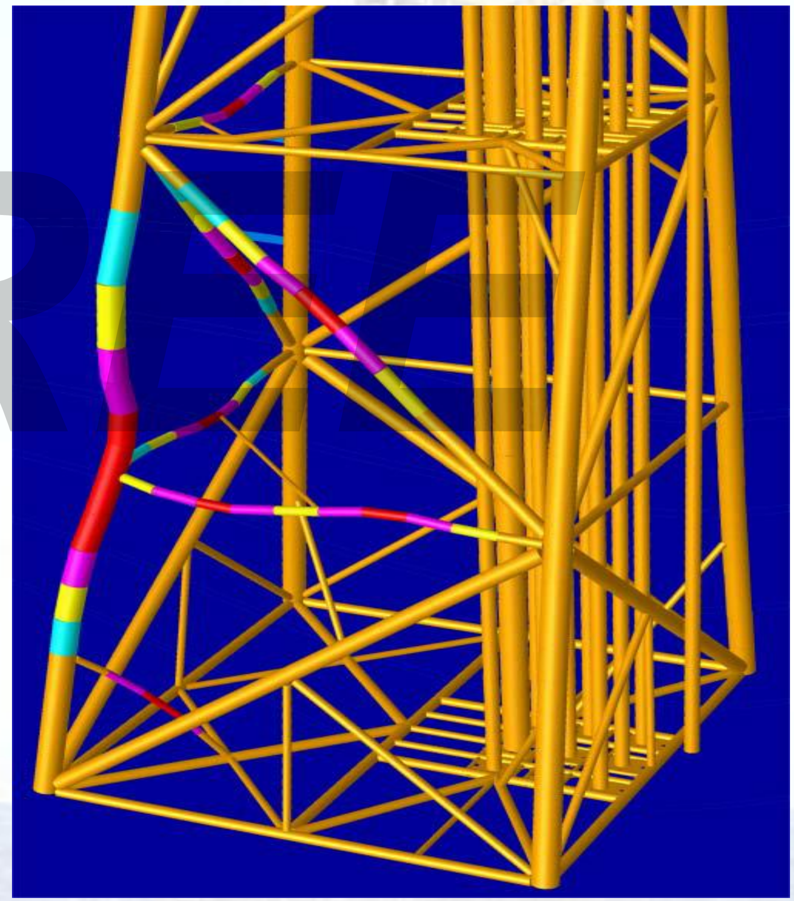
陸域與離岸結構之設計差異-撞擊分析





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離岸風機與離岸鑽油平台基礎設計差異(1/4)

- 離岸風機結構通常沒有載人，因此設計標準較鑽油平台低。但在歐洲有少部分業主為減少預算，採用非離岸用鋼，如EN 10025建築用鋼或 EH36船舶用鋼。此將造成高失敗風險。

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離岸風機基礎與離岸鑽油平台設計差異(2/4)

- 歐洲大部分採用單樁式(monopile)基礎，結構系統無贅餘度，承擔高破壞風險。鑽油平台一般採用套筒式(jacket)基礎，通常部分構件破壞，尚不致影響結構安全。

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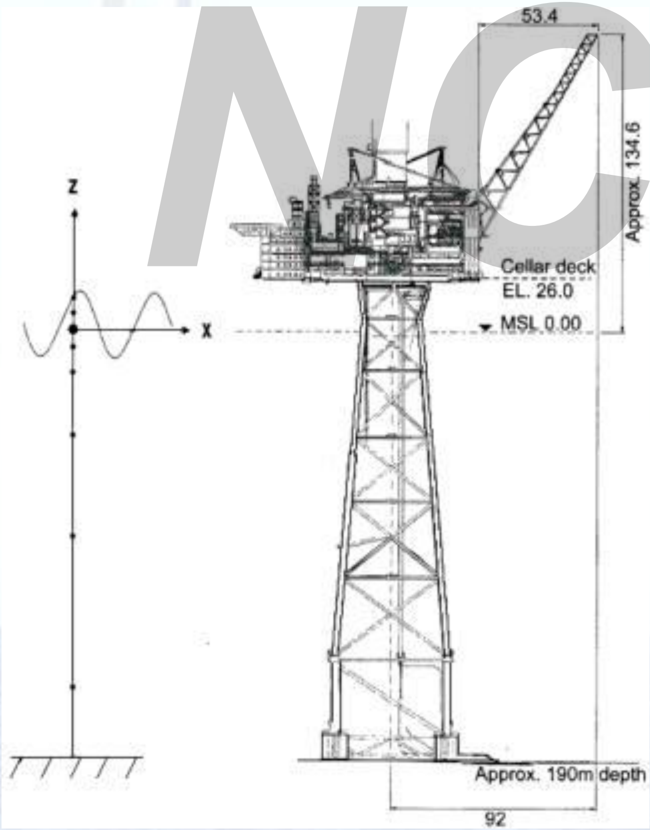
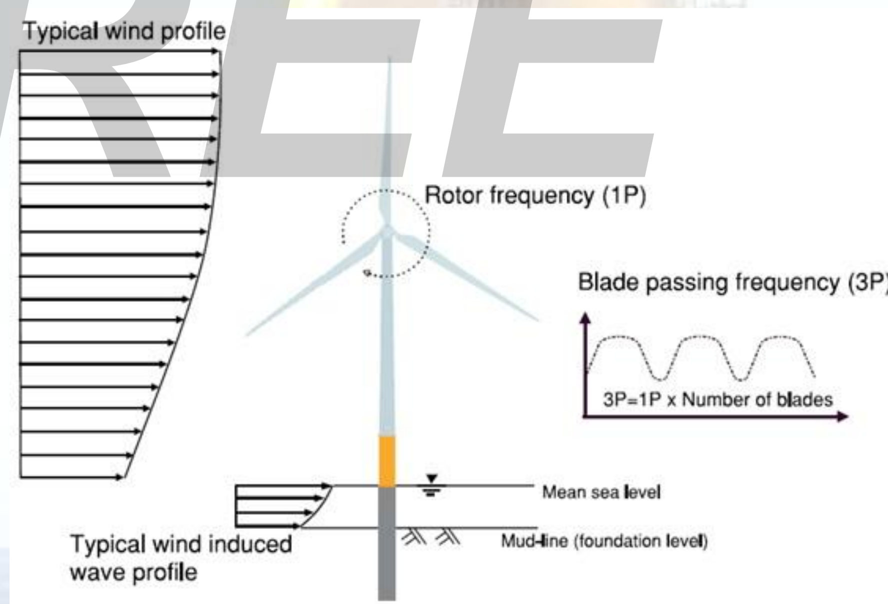
離岸風機與離岸鑽油平台基礎設計差異(3/4)

- 鑽油平台主要承受波浪與海流產生之疲勞效應，而離岸風機除承受前述兩種外力外，風力造成之疲勞效應通常比前述兩種外力更大，因此DNV減少容許鋼板厚度。





離岸風機與離岸鑽油平台基礎設計差異(4/4)

DNVGL-ST-0126

Table 4-4 Thickness limitations (mm) of structural steels for different structural categories and design temperatures (ºC)

Structural Category	Strength Group	Test Temperature (ºC)	Maximum thickness at stated design temperature ≥ 10ºC	0ºC	−10ºC	−20ºC
Secondary	NS	+20	30	30	25	20
		0	50	50	40	30
		-20	100	100	80	60
		-40	150	150	150	150
	HS	0	50	50	40	30
		-20	100	100	80	60
		-40	150	150	150	150
		-60	150	150	150	150
	EHS	0	60	60	50	40
		-20	150	150	100	80
		-40	150	150	150	150
		-60	150	150	150	150
Primary	NS	+20	20	20	10	N.A.
		0	25	25	20	15
		-20	50	50	40	30
		-40	100	100	80	60
	HS	0	25	25	20	15
		-20	50	50	40	30
		-40	100	100	80	60
		-60	150	150	150	150
	EHS	0	30	30	25	20
		-20	60	60	50	40
		-40	150	150	100	80
		-60	150	150	150	150
Special	NS	-20	25	25	20	15
		-40	50	50	40	30
	HS	0	10	10	N.A.	N.A.
		-20	25	25	20	15
		-40	50	50	40	30
		-60	100	100	80	60
	EHS	0	15	15	10	N.A.
		-20	30	30	25	20
		-40	60	60	50	40
		-60	150	150	100	80

N.A. = no application

DNV-OS-C101

Table 5 Thickness limitations (mm) of structural steels for different structural categories and service temperatures (ºC)

Structural Category	Strength group	Grade	≥ 10	0	-10	-20	-25	-30
Secondary	NS	A	35	30	25	20	15	10
		B/BW	70	60	50	40	30	20
		D/DW	150	150	100	80	70	60
		E/EW	150	150	150	150	120	100
	HS	A/AW	60	50	40	30	20	15
		D/DW	120	100	80	60	50	40
		E/EW	150	150	150	150	120	100
		F	150	150	150	150	*)	*)
	EHS	A	70	60	50	40	30	20
		D/DW	150	150	100	80	70	60
		E/EW	150	150	150	150	120	100
		F	150	150	150	150	*)	*)
Primary	NS	A	30	20	10	N.A.	N.A.	N.A.
		B/BW	40	30	25	20	15	10
		D/DW	70	60	50	40	35	30
		E/EW	150	150	100	80	70	60
	HS	A/AW	30	25	20	15	12.5	10
		D/DW	60	50	40	30	25	20
		E/EW	120	100	80	60	50	40
		F	150	150	150	150	*)	*)
	EHS	A	35	30	25	20	17.5	15
		D/DW	70	60	50	40	35	30
		E/EW	150	150	100	80	70	60
		F	150	150	150	150	*)	*)
Special	NS	D/DW	35	30	25	20	17.5	15
		E/EW	70	60	50	40	35	30
	HS	A/AW	15	10	N.A.	N.A.	N.A.	N.A.
		D/DW	30	25	20	15	12.5	10
		E/EW	60	50	40	30	25	20
		F	20	100	80	60	50	40
	EHS	A	20	15	10	N.A.	N.A	N.A.
		D/DW	35	30	25	20	17.5	15
		E/EW	70	60	50	40	35	30
		F	150	150	100	80	70	60

*) For service temperature below -20ºC, the limit tobe specially considered.

N.A. = no application

In order to obtain thickness limitation for intermediate service temperature, linear interpolation may be used.

離岸風機常見與基礎型式 - 單樁式

離岸風機常見與基礎型式 - 套筒式

海上變電站

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- 耐震設計議題

離岸風機/變電站支撐結構設計系列標準

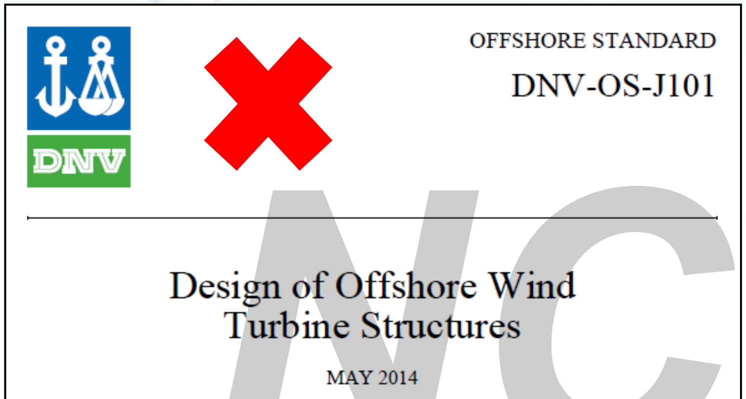
- 離岸風機支撐結構
  - DNVGL-ST-0126 : Support structures for wind turbines
  - DNVGL-ST-0437 : Loads and site conditions for wind turbines
  - DNVGL-RP-0416: Corrosion protection for wind turbines

- 海上變電站
  - DNVGL-ST-0145: Offshore Substations
  - DNVGL-RP-C205: Environmental conditions and Environmental loads
  - DNVGL-OS-C101: Design of offshore steel structures, general - LRFD method

- 共同
  - DNVGL-OS-B101: Metallic materials
  - DNVGL-OS-C401: Fabrication and testing of offshore structures
  - DNVGL-RP-C203 : Fatigue design of offshore steel structures
  - DNV-RP-B401: Cathodic protection design

風機支撐結構設計標準DNVGL-ST-0126

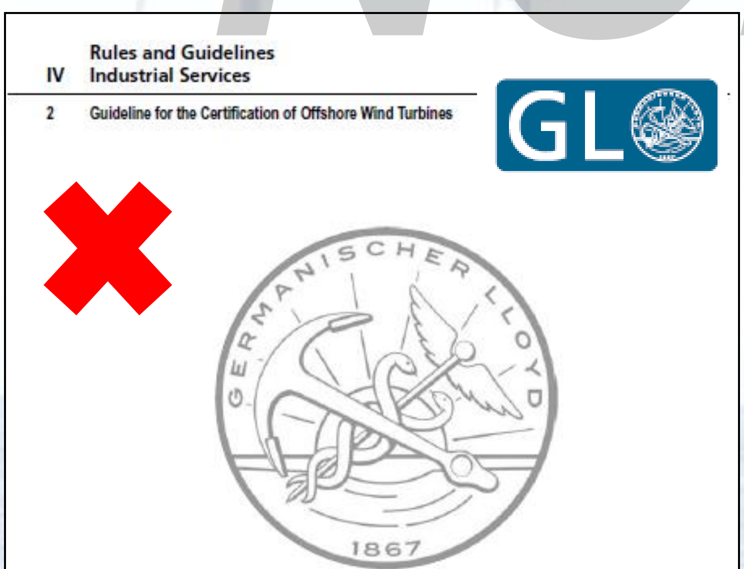
DNV-OS-J101 + GL Guideline = DNVGL-ST-0126


OFFSHORE STANDARD

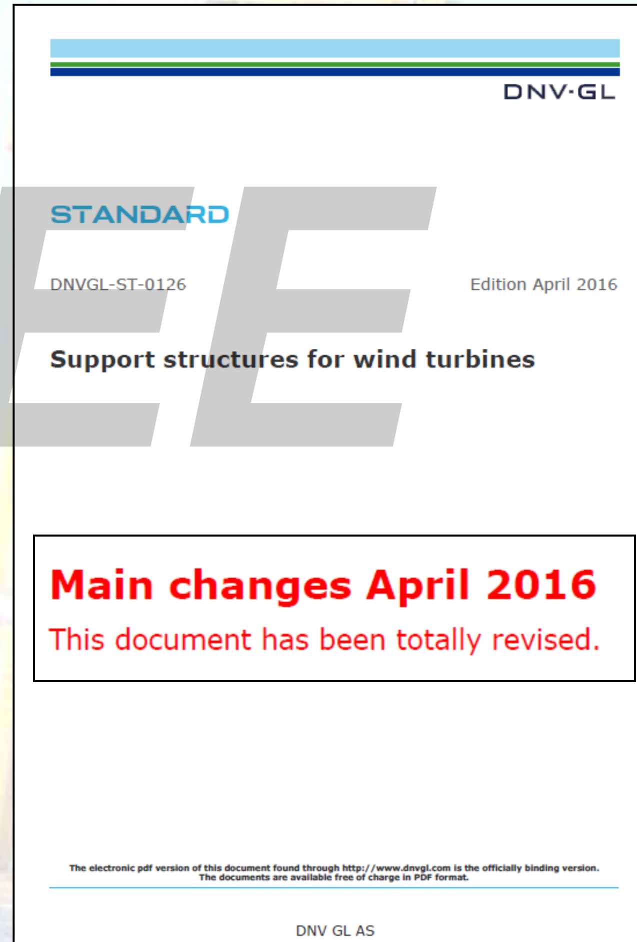
DNV-OS-J101

Design of Offshore Wind Turbine Structures

MAY 2014


IV Rules and Guidelines
Industrial Services

2 Guideline for the Certification of Offshore Wind Turbines


DNV·GL

STANDARD

DNVGL-ST-0126 Edition April 2016

Support structures for wind turbines

Main changes April 2016

This document has been totally revised.

The electronic pdf version of this document found through http://www.dnvgl.com is the officially binding version.
The documents are available free of charge in PDF format.

DNV GL AS

ABS風機支撐結構設計導引

GUIDE FOR BUILDING AND CLASSING

BOTTOM-FOUNDED OFFSHORE WIND TURBINE INSTALLATIONS

OCTOBER 2015 (Updated March 2018 – see next page)

American Bureau of Shipping
Incorporated by Act of Legislature of
the State of New York 1862

ABS Plaza
16855 Northchase Drive
Houston, TX 77060 USA

DNVGL-ST-0126 風機支撐結構設計標準

- 目的:
  - DNVGL-ST-0126為離岸風機支撐結構之設計標準 ，但也包括材料、執行、運維等內容。

- 內容包含:
  - 設計原理
  - 鋼結構設計與防蝕
  - 混凝土結構設計
  - 灌漿連接
  - 地工設計
  - 淘刷保護
  - 運維需求

設計原理

2.1 Introduction

2.1.1 General

2.1.1.1 This section describes design principles and design methods for wind turbine structural design, including:

— design by partial safety factor method　**LRFD法**

— design assisted by testing

— probability-based design.

$$S_d = \sum_{i=1}^{n} \gamma_{fi} S_{ki} < R_d = \frac{R_k}{\gamma_m}$$

載重　強度

2.5.2.1 The safety level of a structure or a structural component is considered to be satisfactory when the design load effect S_d does not exceed the design resistance R_d:

$$S_d \le R_d$$

This is the design criterion. The design criterion is also known as the design inequality. The corresponding equation $S_d = R_d$ forms the design equation.

Table 4-2 Partial safety factors for loads γ_F

Functional and environmental loads					Permanent loads*		
ULS		FLS	ALS	SLS	ULS		FLS, ALS, SLS
Normal	Abnormal				Favourable	Unfavourable	
N 1,35***	A 1,1	F 1,0	F 1,0	F 1,0	0,9**	1,1**	1,0

* Permanent loads include dead loads and pretension loads for the support structure design.

For submerged sub-structures, for example a GBS placed on the seabed, the Permanent load is the total weight minus the buoyancy determined at the still water level

** Factors for permanent loads in ULS may be taken as 1,0 if appropriate measures are taken.

*** For DLC 1.1 the partial load factor shall be γ_F = 1,25; for DLC 2.5 the partial load factor shall be γ_F = 1,2

The following formulation according to IEC 61400-1, Table 3 may be applied:

If for normal design situations the characteristic value of the load response $F_{gravity}$ due to gravity may be calculated for the design situation in question, and gravity is an unfavourable load, the partial load factor for combined loading from gravity and other sources may have the value

$$\gamma_f = 1{,}1 + \varphi\varsigma^2 \text{ and } \varphi = \begin{cases} 0{,}15 \text{ for DLC 1.1} \\ 0{,}25 \text{ otherwise} \end{cases}$$

$$\zeta = \begin{cases} 1 - \left|\frac{F_{gravity}}{F_k}\right| ; & |F_{gravity}| \le |F_k| \\ 0; & |F_{gravity}| > |F_k| \end{cases}$$

where F_k = characteristic value for loads

4.5.1.5 If NORSOK N-004 is applied, the material factor for tubular members shall be taken as

$$\gamma_M = \begin{cases} 1.10 & \text{for } \bar{\lambda}_s < 0.5 \\ 0.80 + 0.60\bar{\lambda}_s & \text{for } 0.5 \le \bar{\lambda}_s \le 1.0 \\ 1.40 & \text{for } \bar{\lambda}_s > 1.0 \end{cases}$$

where $\bar{\lambda}_s$ is the weighted relative slenderness relevant for local buckling under combined axial, bending and hoop stresses determined according to NORSOK N-004 Section 6.3.7.

極限狀態(DNVGL-ST-0126)

2.4.1.2 The following limit states are considered in this standard:

Ultimate limit states (ULS) correspond to the maximum load-carrying resistance

Fatigue limit states (FLS) correspond to failure due to the effect of dynamic loading

Accidental limit states (ALS) correspond to (1) maximum load-carrying capacity for (rare) accidental loads or (2) post-accidental integrity for damaged structures.

Serviceability limit states (SLS) correspond to tolerance criteria applicable to normal use.

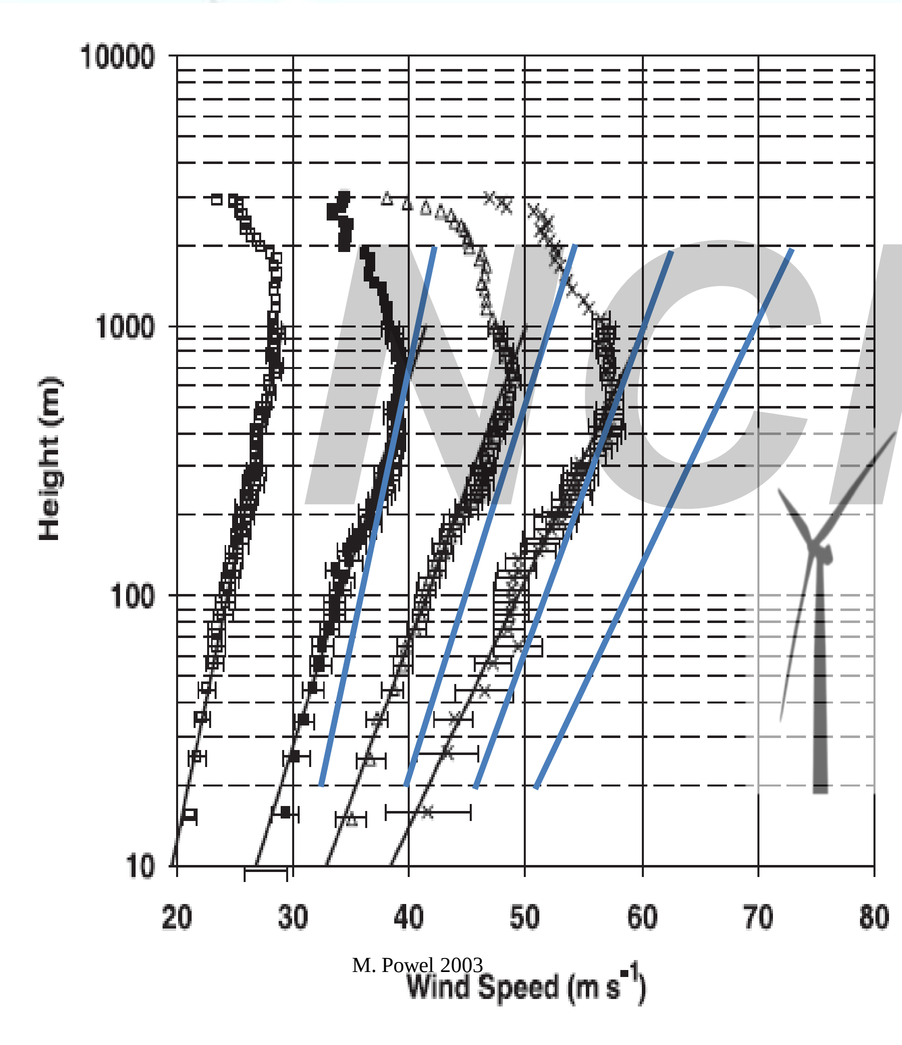
環境條件(DNVGL-ST-0437)

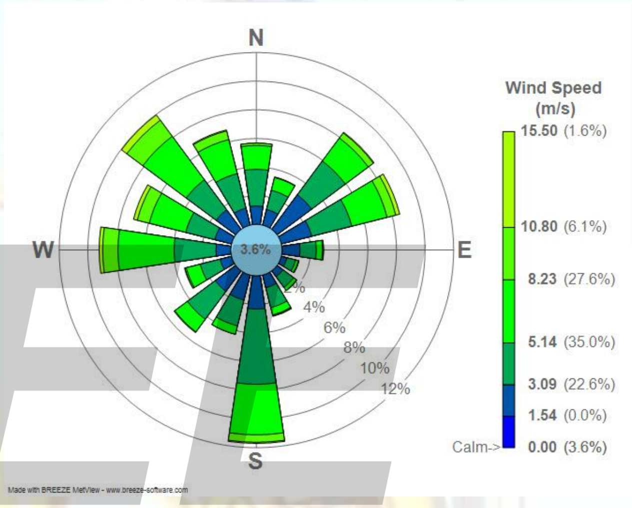
- 平均海水面(MSL)
- 潮位範圍(LAT and HAT)
- 暴潮(1年與50年迴歸期)
- 最高水位(HSWL)
- 最低水位(LSWL)
- 最高水位高程
- 海水密度
- 海水溫度變化
- 鹽度
- 示性波高(1年與50年迴歸期)
- 設計波高(1年與50年迴歸期)
- 設計波浪周期範圍(1年與50年迴歸期)
- 流速(1年與50年迴歸期)
- 波譜與參數
- 肯定(deterministic)波浪模型與參數
- 碎波模型與參數(高度與週期)
- 海冰發生性 (每年與極端海冰厚度、強度，互制...等)
- 附著海生物厚度、密度

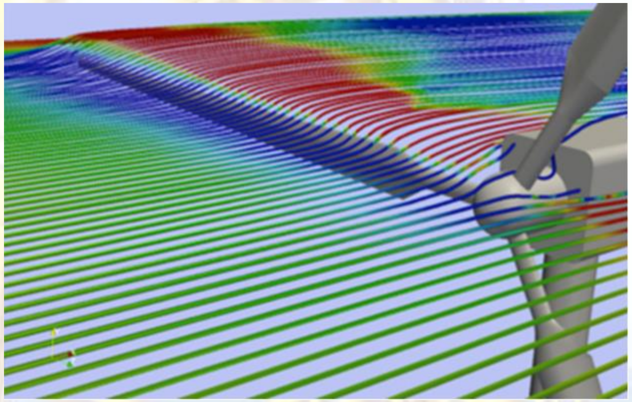
氣動力

- 風速剖面分布

主要疲勞來源之一





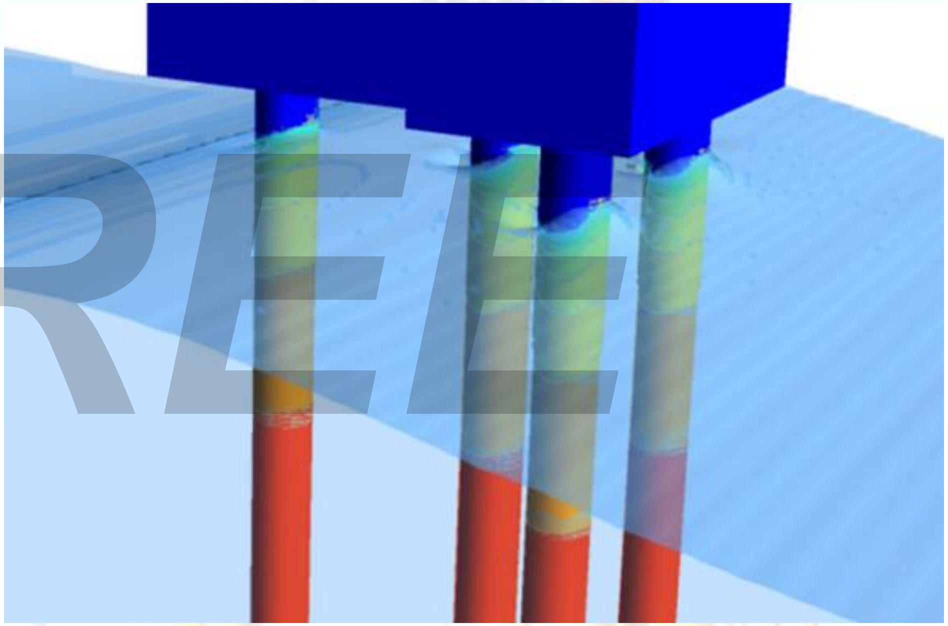


水動力

- 波浪
- 海流
- 附著海生物

主要疲勞來源之一

Period → Height ↓	0 – 2	2 – 4	4 - 6	6 - 8	8 - 10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	Total	%	Tmed
0.0-0.5	85171	166779	50632	10991	2167	425	110	25	13	7	5	0	316325	17.27	3.31
0.5-1.0	11867	209032	229458	111698	40289	11925	2982	547	132	49	17	0	616786	33.70	5.38
1.0-1.5	322	47777	164356	145537	73169	27184	7458	1491	271	55	32	0	467642	25.53	6.91
1.5-2.0	6	6545	64094	85930	53460	23926	7570	1592	266	41	25	0	243454	13.30	7.77
2.0-2.5	0	742	19753	38224	27786	14938	5669	1263	190	37	13	2	108616	6.93	8.41
2.5-3.0	0	82	5311	14769	12522	8057	3290	798	149	11	3	0	44992	2.46	8.96
3.0-3.5	0	6	1277	5433	5522	4149	1907	510	90	10	3	0	18907	1.03	9.50
3.5-4.0	0	1	309	2006	2350	1963	987	265	51	3	0	0	7935	.43	9.90
4.0-4.5	0	0	69	733	1117	973	535	168	33	1	0	0	3629	.20	10.32
4.5-5.0	0	0	12	234	513	507	321	60	11	0	0	0	1658	.09	10.68
5.0-5.5	0	0	4	83	230	271	165	52	6	0	0	0	811	.04	11.31
5.5-6.0	0	0	1	30	115	123	86	23	1	0	0	0	379	.02	10.98
6.0-6.5	0	0	0	8	39	69	35	12	0	0	0	0	163	.00	12.14
6.5-7.0	0	0	0	3	21	26	16	2	0	0	0	0	68	.00	11.16
7.0-7.5	0	0	0	0	5	18	15	2	0	0	0	0	40	.00	11.93
7.5-8.0	0	0	0	2	3	3	7	0	0	0	0	0	15	.00	11.60
8.0-8.5	0	0	0	0	2	3	1	0	0	1	0	0	7	.00	10.34
8.5-9.0	0	0	0	0	0	2	2	0	0	0	0	0	4	.00	12.00
9.0-9.5	0	0	0	0	1	2	1	0	0	0	0	0	4	.00	11.25
9.5-10.0	0	0	0	0	1	1	2	0	0	0	0	0	4	.00	11.50
10.0-10.5	0	0	0	0	0	2	0	0	0	0	0	0	2	.00	13.00
10.5-11.0	0	0	0	0	0	0	0	0	0	0	0	0	.00	-	-
11.0-11.5	0	0	0	0	0	0	0	0	0	0	0	0	.00	-	-
11.5-12.0	0	0	0	0	0	1	0	0	0	0	0	0	1	.00	12.00
Total	97368	429864	535276	415681	219311	94475	31041	6800	1213	215	98	2	1831442	100.00	
%	5.32	23.48	29.23	22.70	11.97	5.16	1.69	.37	.07	.01	.00	.00			
Mean Height	.31	.64	1.06	1.39	1.60	1.81	2.01	2.10	2.02	1.62	1.49	2.17			



$$F = \underbrace{\rho C_m V \dot{u}}_{F_I} + \underbrace{\frac{1}{2}\rho C_d A u|u|}_{F_D},$$

海生物附著厚度(DNVGL-ST-0437 v.s. ABS)

1. DNVGL-ST-0437

Guidance note:

Unless data indicate otherwise, the following marine growth profile may be used for design in Norwegian and UK waters:

Depth below MWL (m)	Marine growth thickness (mm)	
	Central and Northern North Sea (56° to 59° N)	Norwegian Sea (59° to 72° N)
−2 to 40	100	60
>40	50	30

Somewhat higher values, up to 150 mm between sea level and LAT −10 m, may be seen in the Southern North Sea.

For the Baltic Sea, 100 mm should be considered below -2 m MSL unless data indicate otherwise.

Offshore central and southern California, marine growth thicknesses of 200 mm are common.

In the Gulf of Mexico, the marine growth thickness may be taken as 38 mm between LAT+3 m and 50 m depth, unless site-specific data and studies indicate otherwise.

Offshore West Africa, the marine growth thickness may be taken as 100 mm between LAT and 50 m depth and as 300 mm in the splash zone above LAT, unless data indicate otherwise.

The values given should be understood as hard growth equivalent value, i.e. it may be hard growth or a thicker layer of soft growth, which has approximately the same effect as the given thickness of hard growth.

The outer diameter of a structural member subject to marine growth should be increased by twice the recommended thickness of the marine growth at the location in question.

The type of marine growth may have an impact on the values of the hydrodynamic coefficients that are used in the calculations of hydrodynamic loads from waves and current.

2. ABS

7 Marine Growth

Marine growth is to be considered in the design of the Bottom-founded Offshore Wind Turbine Installation. Estimates of the rate and extent of marine growth may be based on past experience and available field data. Particular attention is to be paid to increases in hydrodynamic loading due to increased diameters and surface roughness of members caused by marine fouling as well as to the added weight and increased inertial mass of submerged structural members. The types of fouling likely to occur and their possible effects on corrosion protection coatings are to be considered.

設計地震
(DNV-OS-J101 v.s. DNVGL-ST-0437)

DNV-OS-J101

H 100 Seismicity

101 The level of seismic activity of the area where the wind turbine structure is to be installed shall be assessed on the basis of previous record of earthquake activity as expressed in terms of frequency of occurrence and magnitude.

102 For areas where detailed information on seismic activity is available, the seismicity of the area may be determined from such information.

103 For areas where detailed information on seismic activity is not available, the seismicity is to be determined on the basis of detailed investigations, including a study of the geological history and the seismic events of the region.

104 If the area is determined to be seismically active and the wind turbine structure will be affected by an earthquake, an evaluation shall be made of the regional and local geology in order to determine the location and alignment of faults, epicentral and focal distances, the source mechanism for energy release and the source-to-site attenuation characteristics. Local soil conditions shall be taken into account to the extent that they may affect the ground motion. The seismic design, including the development of the seismic design criteria for the site, shall be in accordance with recognised industry practice.

105 The potential for earthquake-induced sea waves, also known as tsunamis, shall be assessed as part of the seismicity assessment.

106 For details of seismic design criteria, reference is made to ISO 19901-2.

DNVGL-ST-0126

7.3.1.7 If the area, where the wind turbine structure is to be installed, is determined to be seismically active and the substructure and foundation will be affected by an earthquake, an evaluation shall be made of the regional and local geology in order to determine the location and alignment of faults, epicentral and focal distances, the source mechanism for energy release and the source-to-site attenuation characteristics. Local soil conditions shall be taken into account to the extent that they may affect the soil motion. The seismic design, including the development of the seismic design criteria for the site, shall be in accordance with recognised industry practice. For details of seismic design criteria, reference is made to ISO 19901-2 and to EN 1998-5

DNVGL-ST-0437

4.2.9 Seismic loads

The analysis of the dynamic response shall be performed using recognized procedures such as:

— response spectrum analysis

— time history analysis.

A three-dimensional model of the structure should be used for the analysis. The combination of earthquake loads with other loads is described in Table 4-4 and [4.6.3]. The seismic response spectrum is generally given in local building codes. When applying the seismic response spectrum, it shall be ensured, that the recurrence period is the same as that the chosen analysis method is based on.

When the response spectrum analysis is applied for the combination of the modal maxima, the use of the complete quadratic combination (CQC) method as described in EN 1998 is recommended.

When time domain simulations are used, the ground acceleration at the surface of the seabed shall be derived from the seismic response spectrum taking into account the soil properties. A sufficient number of stochastic acceleration time series of sufficient duration shall be taken into account.

Further information may be found in IEC 61400-1, 11.6 and the corresponding Annex.

IEC 61400-1

11.6 Assessment of earthquake conditions

There are no earthquake resistance requirements for standard class turbines because such events are only design driving in a few regions of the world. No earthquake assessment analysis is required for sites already excluded by the applicable local seismic code due to their weak seismic action. For locations where the seismic load cases described below are critical, the engineering integrity shall be demonstrated for the wind turbine site conditions. The assessment may be based on Annex C. The evaluation of load shall take account of the combination of seismic loading with other significant, frequently occurring operational loads.

The seismic loading shall depend on ground acceleration and response spectrum requirements as defined in local codes. If a local code is not available or does not give the ground acceleration and response spectrum, an appropriate evaluation of these parameters shall be carried out.

The ground acceleration shall be evaluated for a 475-year recurrence period.

設計地震 - 美國ABS設計導引

Seismicity and Earthquake Related Phenomena

The effects of earthquakes on the Bottom-founded Offshore Wind Turbine Installation located in areas known to be seismically active are to be taken into account.

Levels of Earthquake Conditions *(15 January 2013)*

The magnitudes of the parameters characterizing the earthquakes with return periods appropriate to the design life and the intended safety level of the Bottom-founded Offshore Wind Turbine Installation are to be determined. Two levels of earthquake conditions are to be considered to address the risk of damage and structure collapse, respectively:

i) *Strength Level.* Ground motion which has a reasonable likelihood of not being exceeded at the site during the design life of the Bottom-founded Offshore Wind Turbine Installation

ii) *Ductility Level.* Ground motion for a rare, intense earthquake to be applied to evaluate the risk of

Regional and Site-specific Data

The anticipated seismicity of an area is, to the extent practicable, to be established based on suitable regional and site specific data including the following:

i) Magnitudes and recurrence intervals of seismic events

ii) Proximity to active faults

iii) Type of faulting

iv) Attenuation of ground motion between the faults and the site

v) Subsurface soil conditions

vi) Records from past seismic events at the site where available, or from analogous sites

Other Earthquake Related Phenomena

The seismic data are to be used to establish quantitative Strength Level and Ductility Level earthquake criteria describing the earthquake induced ground motion expected during the design life of the Bottom-founded Offshore Wind Turbine Installation. In addition to ground motion, and as applicable to the site in question, the following earthquake related phenomena are to be taken into account.

i) Liquefaction of subsurface soils submarine slides

ii) Tsunamis

iii) Acoustic overpressure shock waves

11 Earthquake Loads

For the Bottom-founded Offshore Wind Turbine Installation located in seismically active areas, the Strength Level and Ductility Level earthquake induced ground motions (see 3-6/9) are to be determined based on seismic data applicable to the installation site.

Earthquake ground motions are to be described by either applicable ground motion records or response spectra consistent with the return period appropriate to the design life of the Bottom-founded Offshore Wind Turbine Installation. Available standardized spectra applicable to the region of the installation site are acceptable provided such spectra reflect site-specific conditions affecting frequency content, energy distribution, and duration. These conditions include

- The type of active faults in the region
- The proximity of the site to the potential source faults
- The attenuation or amplification of ground motion between the faults and the site
- The soil conditions at the site

The ground motion description used in the design is to consist of three components corresponding to two orthogonal horizontal directions and the vertical direction. All three components are to be applied to the structure simultaneously.

When a standardized response spectrum, such as the one given in the API RP 2A-WSD, is used for structural analysis, input values of ground motion (spectral acceleration representation) are not to be less severe than the following.

- 100% in both orthogonal horizontal directions
- 50% in the vertical direction

When three-dimensional, site-specific ground motion spectra are developed, the actual directional accelerations are to be used. If single site-specific spectra are developed, accelerations for the remaining two orthogonal directions are to be applied in accordance with the factors given above.

If time history method is used for structural analysis, at least three sets of ground motion time histories are to be employed. The manner in which the time histories are used is to account for the potential sensitivity of the structure's response to variations in the phasing of the ground motion records.

Structural appurtenances and turbine equipment are to be designed to resist earthquake induced accelerations at their foundations.

As appropriate, effects of soil liquefaction, shear failure of soft mud and loads due to acceleration of the hydrodynamic added mass by the earthquake, submarine slide, tsunamis and earthquake generated acoustic shock waves are to be taken into account.

荷重組合(DNVGL-ST-0437)

Table 4-3 Design load cases

Table 4-3 Design load cases (Continued)

Table 4-3 Design load cases (Continued)

Design Situation	DLC	Wind Condition	Marine Condition: Waves	Marine Condition: Wind and wave directionality	Marine Condition: Sea Currents	Marine Condition: Water Level	Other Conditions:	Type of Analysis: Onshore	Type of Analysis: Offshore	Partial safety factor
Drifting sea ice (power production)	9.1	NTM	No waves	n/a	NCM	NWLR	Ice load in horizontal direction	-	U	N
Temperature effects (parked, standing still or idling)	10.2	NWP $V_{hub} = V_1$	$H=H_s(V)$	COD, UNI	NCM	MSL	Temperature effects	U	U	N
Earthquake (power production)	11.1	NTM $V_{in} \le V_{hub} \le V_{out}$	$H=H_s(V)$	COD, UNI	NCM	NWLR	Earthquake	U	U	See [4.3.4]
Earthquake plus grid loss (power production)	11.2	NWP $V_{in} \le V_{hub} \le V_{out}$	$H=H_s(V)$	COD, UNI	NCM	NWLR	Earthquake plus grid loss	U	U	See [4.3.4]
Earthquake plus grid loss (parked, standing still or idling)	11.3	NWP $V_{hub} = V_1$	$H=H_s(V)$	COD, UNI	NCM	NWLR	Earthquake plus grid loss	U	U	See [4.3.4]
Wind farm influence (power production)	12.1	NTM $V_{in} \le V_{hub} \le V_{out}$	$H=H_s(V)$	COD, UNI	NCM	NWLR	Wind farm influence	F/U	F/U	F/N

Drifting sea ice (parked, standing still or idling) … $h = h_{50}$ or largest value of moving ice; Dynamic effects from ice loading …

31組類荷重組合 - 考慮不同作用方向、風速、運轉狀態...，可達10000種組合

材料

- 鋼鐵
  - 韌性
  - 強度
  - 防蝕 (鈦，雙相不銹鋼)
- 特高強度水泥砂漿

各標準鋼材之衝擊能量比較

鋼材型號	吸收能量(J)	試驗溫度(℃)
CNS SN400/490	27(縱向)	0
EN 10025 SM355L	31(縱向) 20(橫向)	-40 -40
ABS EH 36	34	-40
API Spec 2MT1	34	0~10
DNV VL EW36	50(縱向) 34(橫向)	-40
EN 10225 S355G5-10+N/M級	50(縱向) 50(橫向)	-40 -40

次要結構用鋼標準: 27J@0 ℃

美國ABS離岸風機結構鋼材之衝擊能量要求

Structural Steel Plates and Shapes

Group	*Specification & Grade*	*Yield Strength*		*Tensile Strength*	
		ksi	*MPa*	*ksi*	*MPa*
I	ASTM A36	36	250	58-80	400-550
	ASTM A131 Grade A (ABS Grade A)	34	235	58-75	400-515
	ASTM A285 Grade C	30	205	55-75	380-515
	ASTM A131 Grades B, D (ABS Grades B, D)	34	235	58-75	400-515
	ASTM A516 Grade 65	35	240	65-85	450-585
	ASTM A573 Grade 65	35	240	65-77	450-530
	ASTM A709 Grade 36T2	36	250	58-80	400-550
	ASTM A131 Grade E (ABS Grade E) (ABS Grade E)	34	235	58-75	400-515
II	ASTM A572 Grade 42	42	290	60 min.	415 min.
	ASTM A572 Grade 50	50	345	65 min.	450 min.
	ASTM A588 (to 2 in. thick)	50	345	70 min.	485 min.
	ASTM A709 Grades 50T2, 50T3	50	345	65 min.	450 min.
	ASTM A131 Grade AH32 (ABS Grade AH32)	46	315	68-85	470-585
	ASTM A131 Grade AH36 (ABS Grade AH36)	51	350	71-90	490-620
	API Spec 2H-Grade 42	42	290	62-80	425-550
	API Spec 2H-Grade 50 (to $2^{1}/_{2}$ in. thick)	50	345	70-90	485-620
	(over $2^{1}/_{2}$ in. thick)	47	325	70-90	485-620
	API Spec 2W-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	API Spec 2Y-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	ASTM A131 Grades DH32, EH32 (ABS Grades DH32, EH32	46	315	68-85	470-585
	Grades DH36, EH36 (ABS Grades DH36, EH36)	51	350	71-90	490-620
	ASTM A537 Class 1 (to $2^{1}/_{2}$ in. thick)	50	345	70-90	485-620
	ASTM A633 Grade A	42	290	63-83	435-570
	Grades C, D	50	345	70-90	480-620
	ASTM A678 (80) Grade A	50	345	70-90	485-620
	ASTM A992	50-65	345-450	65 min.	450 min.
III	ASTM A537 Class 2	60	415	80-100	550-690
	ASTM A633 Grade E	60	415	80-100	550-690
	ASTM A678 (80) Grade B	60	415	80-100	550-690
	API Spec 2W-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	API Spec 2Y-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	ASTM A710-Grade A Class 3 (to 2 in. thick)	75	515	85 min.	585 min.
	(2 in. to 4 in. thick)	65	450	75 min.	515 min.
	(over 4 in. thick)	60	415	70 min	485 min.

TABLE 4
Charpy Toughness Specification for Steels

Group	*Section Size*	*Energy Absorption (Longitudinal)*	
		ft-lb	*Joules*
I	6 mm < t < 19 mm (0.25 in. < t < 0.75 in.)	15	20
	t > 19 mm (0.75 in.)	20	27
II, III	t > 6 mm (0.25 in.)	25	34

Notes

1 Test Temperatures – The following applies for service temperatures down to –30°C (–22°F). For lower service temperatures, test requirements are to be specially considered.

a For structural members and joints whose performance is vital to the overall integrity of the structure and which experience an unusually severe combination of stress concentration, rapid loading, cold working, and restraint, the impact test guidelines of 2-1/Table 4 are to be met at test temperatures as given below.

Group	*Test Temperature*	*Minimum Service Temperature (As determined by 2-1/1.7)*
I, II	30°C (54°F) below Minimum Service Temperature	
III	–40°C (–40°F)	–10°C (32°F)
	–50°C (–58°F)	–10°C (14°F)
	–50°C (–58°F)	–20°C (–4°F)
	–60°C (–76°F)	–30°C (–22°F)

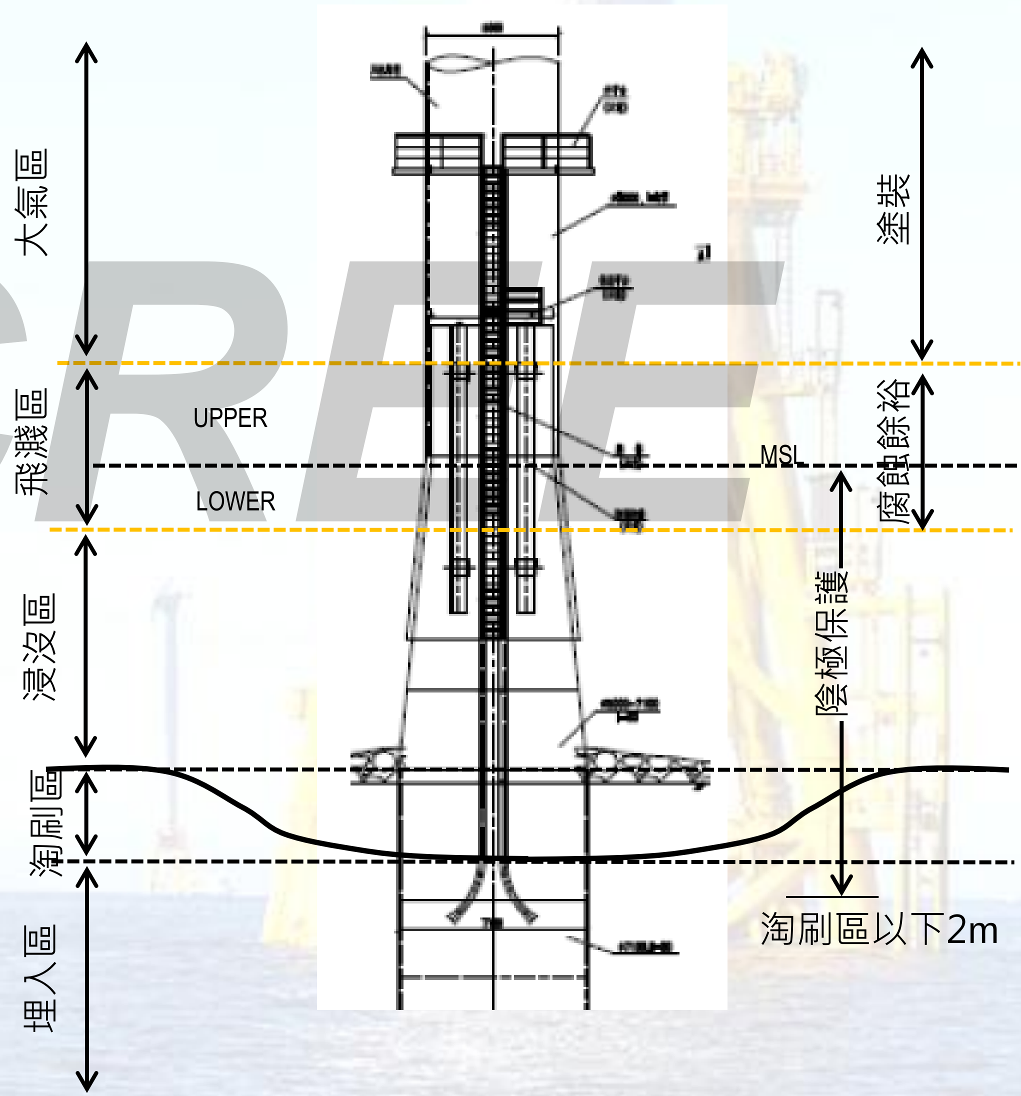
特高強度水泥砂漿

- 抗壓強度:
  - 110~200 MPa (75mm立方試體)
  - 800~1700 kg/cm^2 (300mm*150mm圓柱試體)
- 浸水無收縮

國內能生產?

防蝕策略

- 塗裝系統
- 陰極防蝕
  - 犧牲陽極
  - 外加電流
- 腐蝕餘裕 本土資料?



結構構件強度設計

- 管狀構件強度檢核規範
  - Eurocode 3/EN 1993-1-1 and EN 1993-1-6
  - ISO19902 (D/t < 120)
  - NORSOK N-004 (D/t < 120)

- 節點強度檢核規範
  - ISO19902
  - NORSOK N-004

國內鋼結構設計規範?
AISC?

DNV·GL

STANDARD

DNVGL-ST-0126 Edition April 2016

Support structures for wind turbines

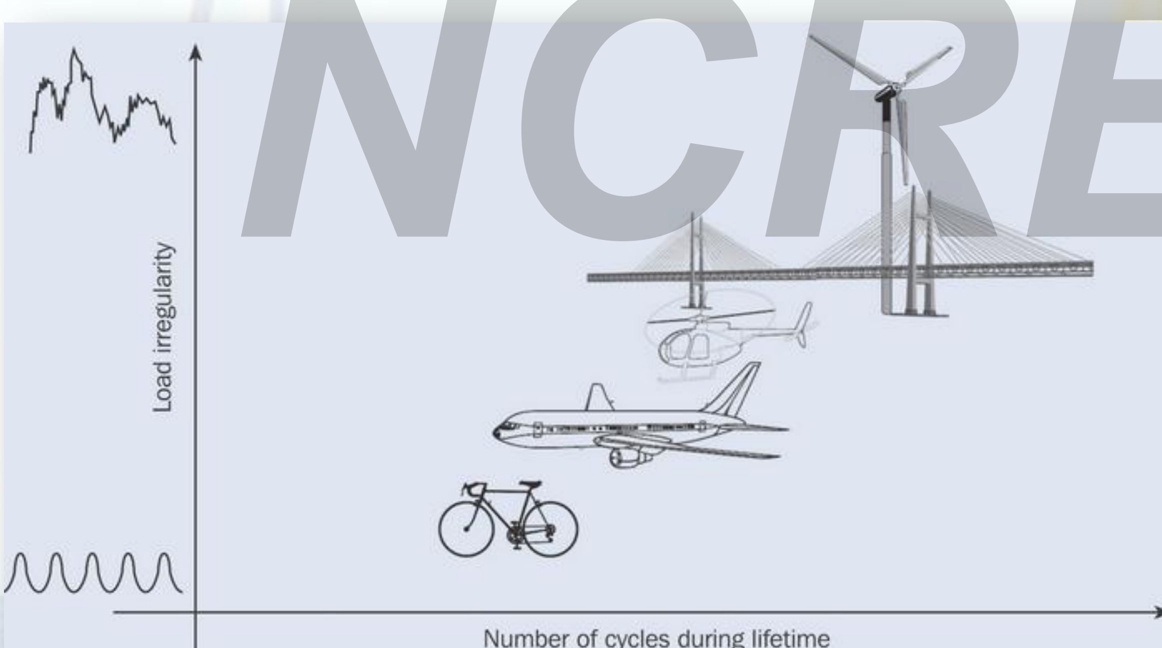
通常採用 →

NORSOK STANDARD **N-004**
Rev. 3, February 2013

Design of steel structures

韌性-疲勞

- 陸上: 車輛
- 海上:風、波、流、機械



非結構設計者熟悉領域

疲勞設計

- 節點應力集中
- 銲接的細微瑕疵->應力集中->細微裂縫->反覆擴大->破裂

應力範圍

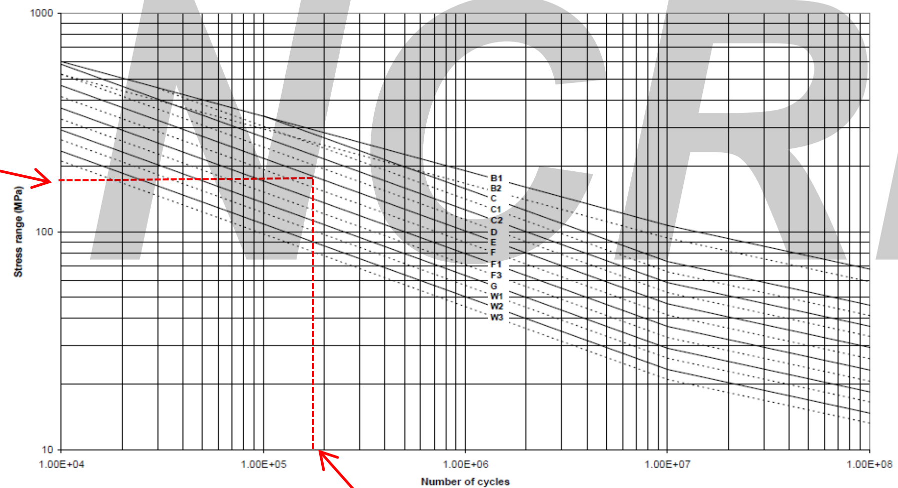
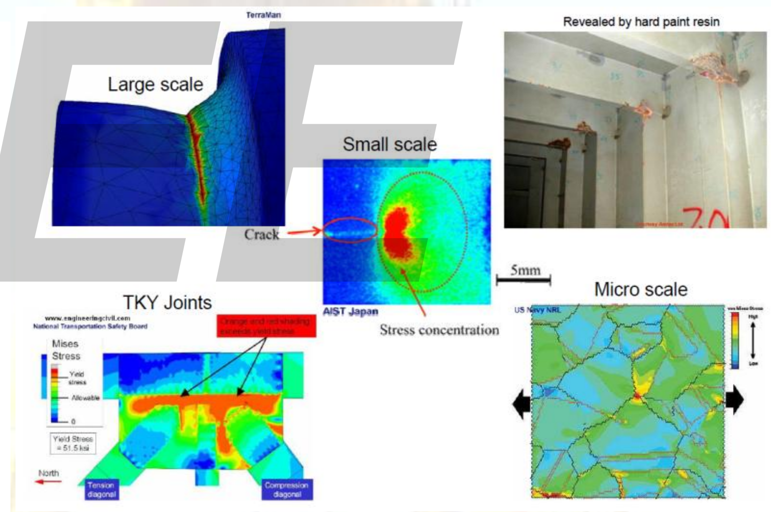


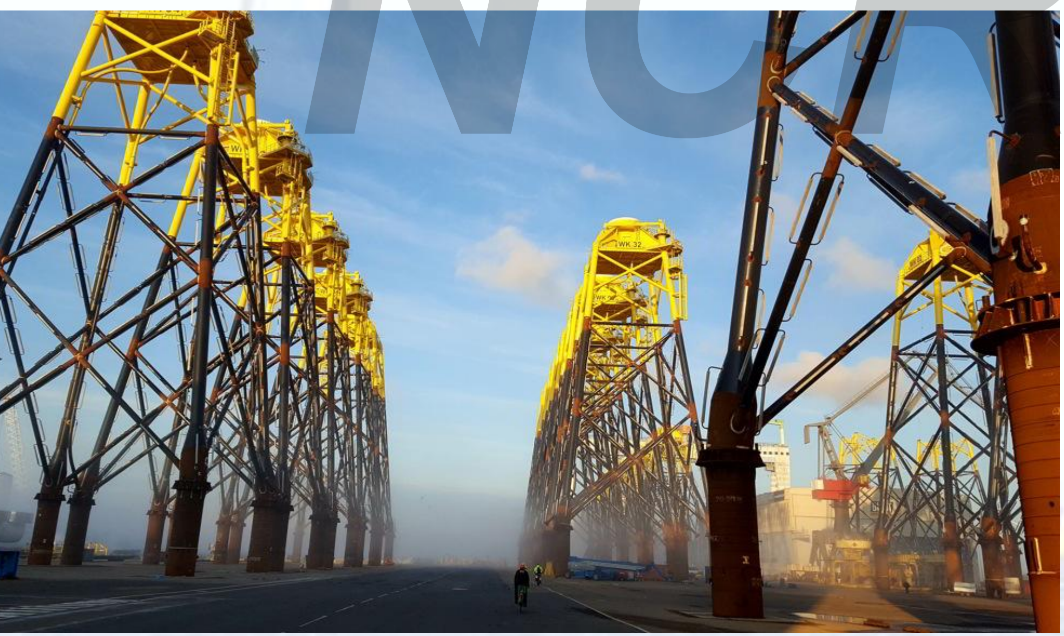
Figure 2-8 S-N curves in air

可承受之破壞次數



設計時需考慮之製造問題

銲工無法進入直徑小於1500mm之管內銲接



組裝場? 存放場? 碼頭?

工作高度>20m

設計時需考慮之製造問題

製造設施限制:

- 製造場地 – 工廠高度、長度?
- 捲管限制– D/t<20?
- 塗裝工廠– 結構能否進出大門? 戶外塗裝，但可能需要修補?
- 裝載能量– 起重機能量? SPMT車輛載重?
- 吊點與結構受力

施工限制因素-運輸與施工

離岸風機支撐結構設計概述

- 離岸結構概觀
- 離岸結構設計考量
- 耐震設計議題

耐震設計規範說明與比較

- **國內建築物耐震設計規範及解說**
  - 建築物高於50m應採動力分析，由於風機高度約於海平面100公尺左右，依其設計理念，應採反應譜或地震歷時進行動力分析

$$\frac{I}{1.4\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m$$

  - 規範要求，三種程度地震須予以考量：
    - 中小度地震(回歸期約為30年) - 結構須保持在彈性限度內，使地震過後，建築物結構體沒有任何損壞
    - 設計地震(回歸期約為475年) - 結構不得產生嚴重損壞，以避免造成嚴重的人命及財產損失，容許建築物在一些特定位置產生塑鉸及
    - 最大考量地震(回歸期約為2500年) - 結構不產生崩塌。

離岸風電設計標準之耐震設計要求

- DNVGL-ST-0126
  - 要求參考ISO 19901-2進行耐震設計
  - DNVGL-ST-0437則建議採用當地規範，否則參考IEC 61400-1、Eurocode 8或API之相關標準，並使用反應譜法或時間歷時分析進行評估。
  - DNVGL-ST-0437標準要求地震相關荷重組合（DLCs 11.1、11.2、11.3）。

耐震設計規範說明與比較

- **IEC 61400-3標準**
  - IEC 61400-3規定參考IEC 61400-1進行耐震設計，
  - 使用當地規範之地表加速度資料及反應譜進行評估，且地表加速度須以475年回歸期進行評估
- **離岸石油氣平台設計標準**
  - DNVGL-OS-C101未說明如何進行耐震設計
  - ISO 19902與API RP-2A WSD皆要求離岸石油氣平台需考量地震效應，且皆將設計地震分為兩個基準：極限基準地震(Extreme Level Earthquake, ELE)及異常基準地震(Abnormal Level Earthquake, ALE)

地震	ISO 19902	API RP-2A WSD
極限基準地震 (ELE)	承受些許損傷或無損傷	足夠強度使結構不發生嚴重損傷
異常基準地震 (ALE)	結構失敗造成死亡及重大環境災害	足夠韌性使結構不發生崩塌

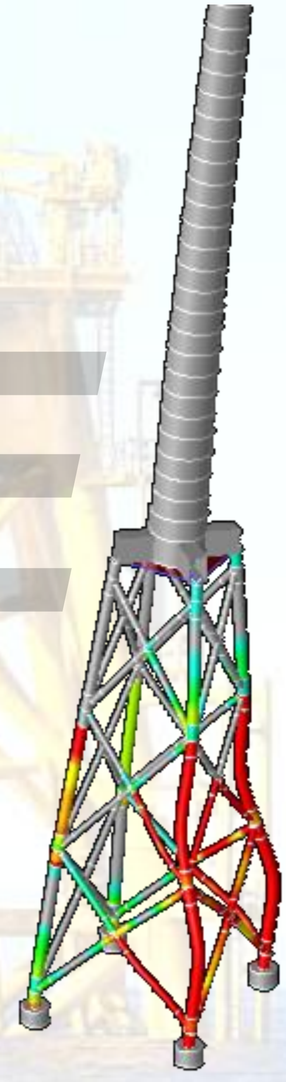
耐震設計規範說明與比較

- 各設計標準之設計地震迴歸期比較

設計標準	設計地震回歸期
建築物耐震設計規範及解說	中小度地震：**30年** 設計地震：**475年** 最大考量地震：**2500年**
DNVGL-ST-0126	ISO 19901-2： **極限基準地震(ELE)與異常基準地震(ALE)**
DNVGL-ST-0437	**475年**
IEC 61400-3	IEC 61400-1：**475年**

套筒式支撐結構韌性分析

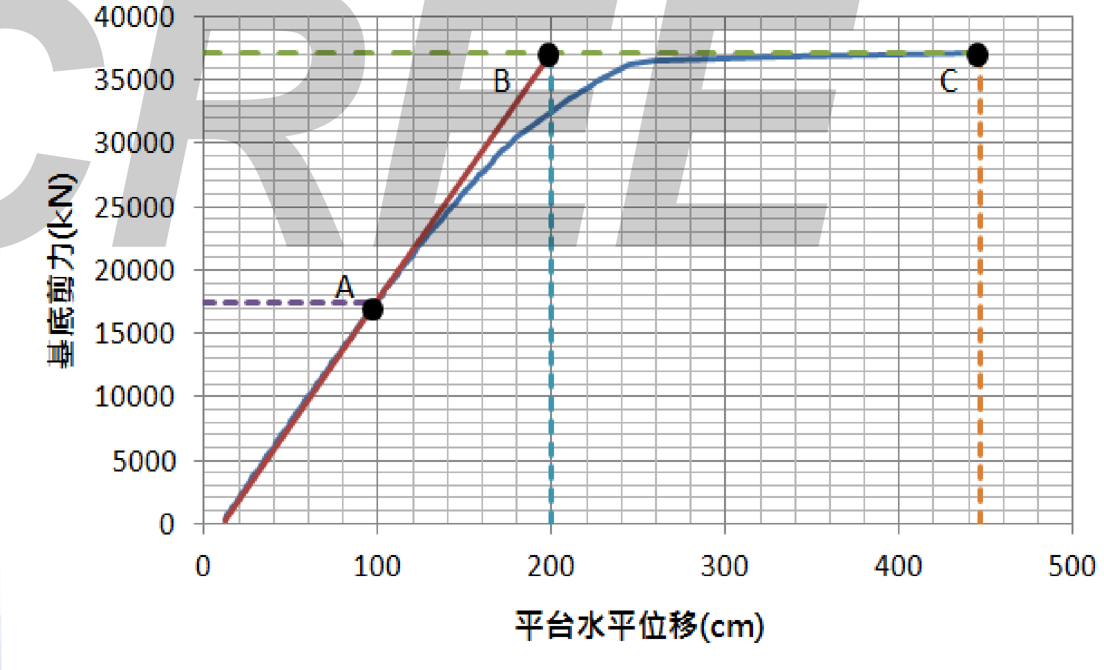
– 以位於水深40m處之套筒式支撐結構為例，探討其韌性容量，並以離岸結構分析軟體SACS 11.0.0.1進行側推分析



結構韌性容量

1. 取得支撐結構在逐步加載下，發生構件開始降伏時，於力-位移曲線圖標記A點。
2. 取得支撐結構在步階加載下，結構崩潰時，於力-位移曲線圖標記C點，其位移為Δ_C。
3. 以A點，沿切線方向與破壞時之基底剪力相交，得B點，其對應之位移為Δ_B。
4. 定義結構韌性容量R為位移Δ_C及Δ_B之比值，如下式所示：

$$韌性容量R = \frac{\Delta_C}{\Delta_B}$$



力與位移關係曲線

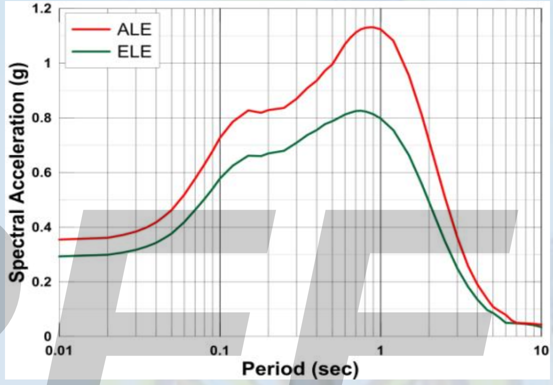
套筒式支撐結構韌性分析-基本假設

土壤

採用彰濱外海一帶之地層，該地層為現代沖積層，屬濁水溪沖積扇之一部分，本區域沉積物以灰色粉土質砂(SM)、灰色低塑性黏土(CL)及粉土(ML)交互出現，偶有含粉土之不良級配砂(SP-SM)、含粉土之級配優良砂偶夾礫石(SP-SM)、含砂之粉土質黏土(CL-ML)及黏土至粉土質砂(SC-SM)。

地震反應譜

依據 ISO 19901-2 推求之設計震譜

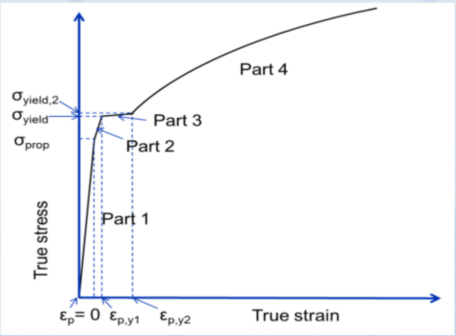


極限基準地震(ELE)之水平向PGA值為0.29g

鋼材

參考DNV-RP-C208-S355鋼材力學特性

項目	參數
厚度[mm]	40< t <63
E [MPa]	210000
σ_{prop} [MPa]	301.9
σ_{yield} [MPa]	336.9
σ_{yield2} [MPa]	345.7
ε_{p_yl}	0.004
ε_{p_y2}	0.015
K[MPa]	725
n	0.166



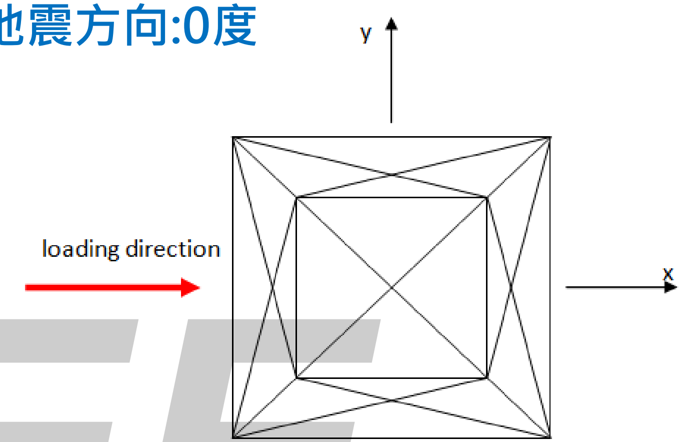
模型幾何

項目	參數	分析模型示意
風機裝置容量	8MW	
風機重量	430 tonnes	
輪轂高度	104m (TVD)*	
設計水深	40m	
土層性質	SM、CL、ML互層	
液化潛能	20 m	
結構型式	套筒式	
套筒腳數目	4隻	
套筒底部寬度	20 m	
套筒頂部寬度	10 m	
樁基礎直徑	3.3 m	
樁基礎長	65 m	

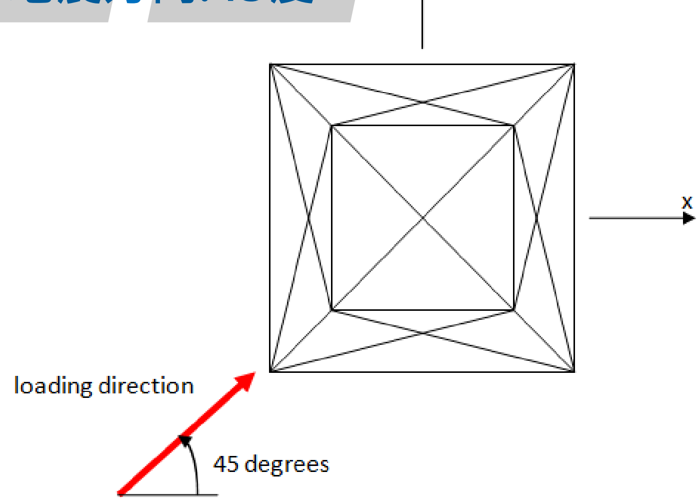
套筒式支撐結構韌性分析-分析案例

地震方向	編號	情境
0度	Case 1	樁土互制，土壤無液化，無風波流作用
	Case 2	樁土互制，土壤液化，無風波流作用
	Case 3	樁土互制，土壤無液化，有風波流作用
	Case 4	樁土互制，土壤液化，有風波流作用
	Case 5	基底固定，有風波流作用
45度	Case 6	樁土互制，土壤無液化，無風波流作用
	Case 7	樁土互制，土壤液化，無風波流作用
	Case 8	樁土互制，土壤無液化，有風波流作用
	Case 9	樁土互制，土壤液化，有風波流作用
	Case 10	基底固定，有風波流作用

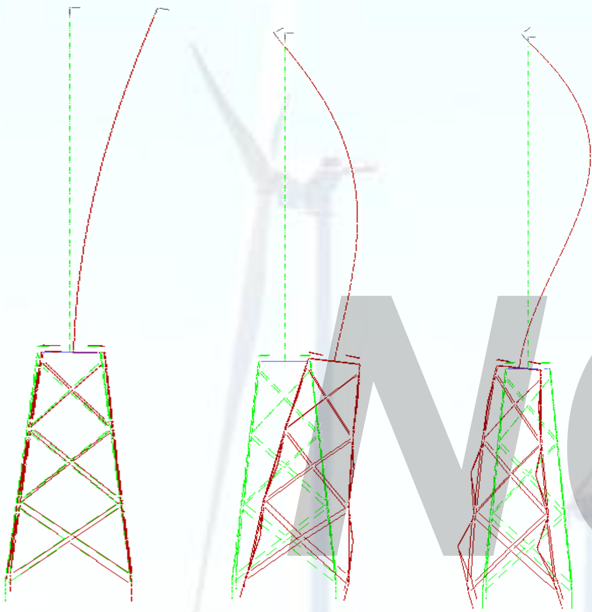
地震方向:0度



地震方向:45度



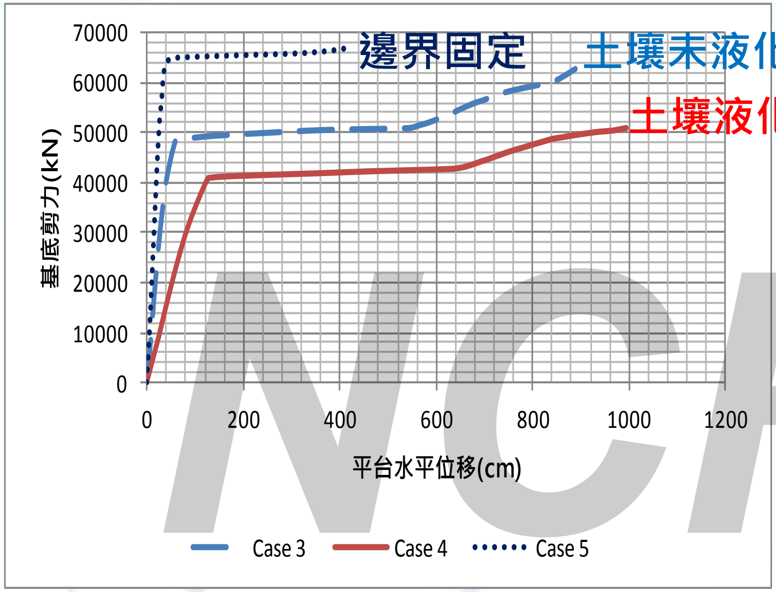
成果比較-模態、韌性容量、動態特性



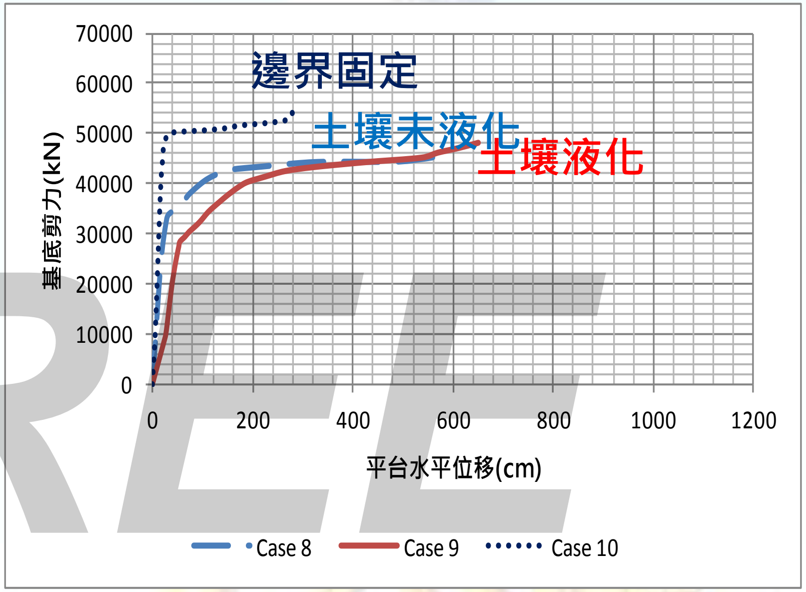
編號	環境條件	Δy (cm)	Δu (cm)	R	第一模態		第二模態	
					週期 (sec)	質量參與因子 (%)	週期 (sec)	質量參與因子 (%)
Case 1	無液化 無風浪	77.8	888.3	11.4	3.29	27.6	0.55	48.1
Case 2	液化 無風浪	115.9	721.1	6.2	3.40	33.8	1.09	64.6
Case 3	無液化 有風浪	78.8	889.5	11.2	3.29	24.5	0.59	55.1
Case 4	液化 有風浪	130.4	992.8	7.6	3.42	32.2	1.23	66.5
Case 5	固定基底 有風浪	32.1	407.4	12.6	3.24	24.5	0.45	31.7
Case 6	無液化 無風浪	47.6	573.2	12.0	3.29	27.6	0.55	47.9
Case 7	液化 無風浪	78.2	718.1	9.1	3.39	33.5	1.08	64.4
Case 8	無液化 有風浪	45.8	593.5	12.9	3.32	28.1	0.59	47.4
Case 9	液化 有風浪	75.4	649.7	8.6	3.43	29.9	1.14	51.9
Case 10	固定基底 有風浪	22.4	281.4	12.5	3.24	24.5	0.45	31.7

- 土壤液化造成結構自然週期增加，韌性容量較低，且對第二模態週期有顯著影響
- 本研究分析案例之第二模態佔主要參與質量
- 土壤發生液化，結構主要週期對應較高之擬加速度反應，即承受較大地震作用力

成果比較-不同束制條件之韌性容量



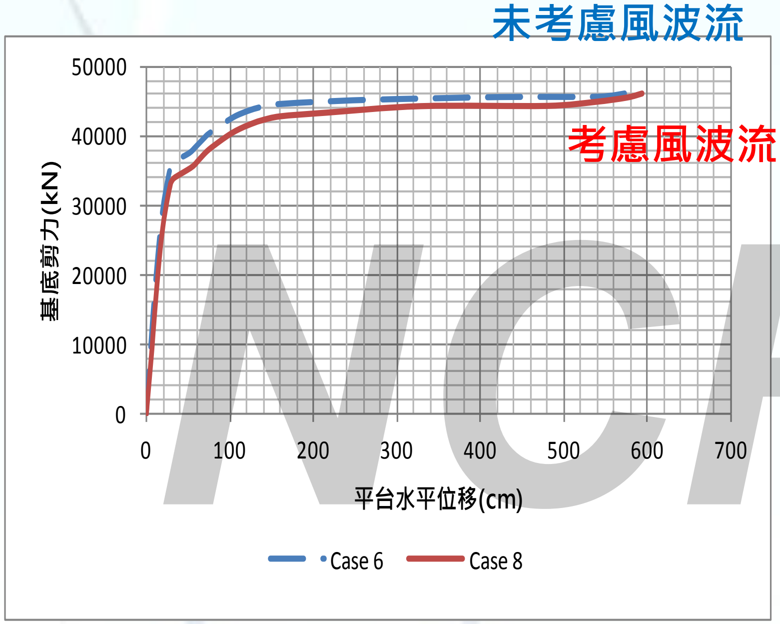
不同束制條件之韌性容量(0度)



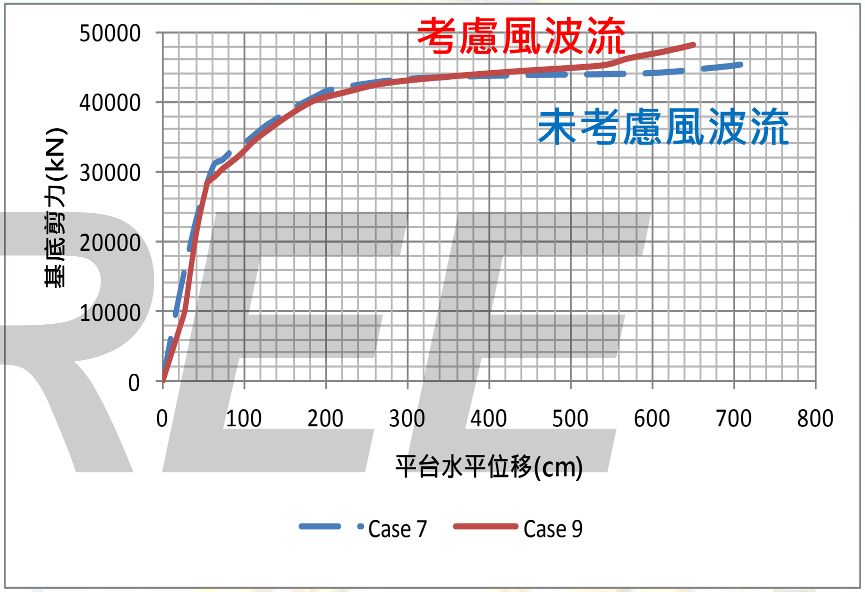
不同束制條件之韌性容量(45度)

土壤束制愈低，其初始斜率愈低，即變位量愈大；
相對地，基底固定的邊界條件，結構整體勁度較高，初始斜率亦較高。

成果比較-不同束制條件之韌性容量



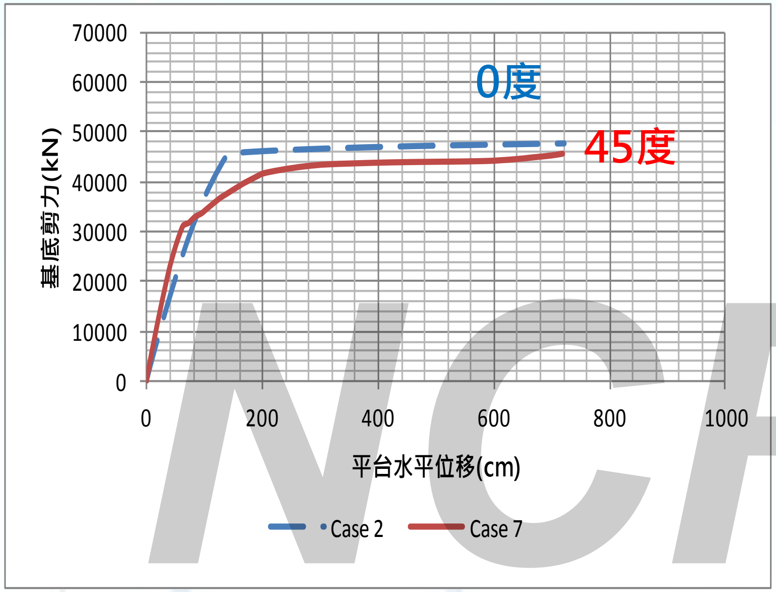
不同加載條件之韌性容量(無液化土層)



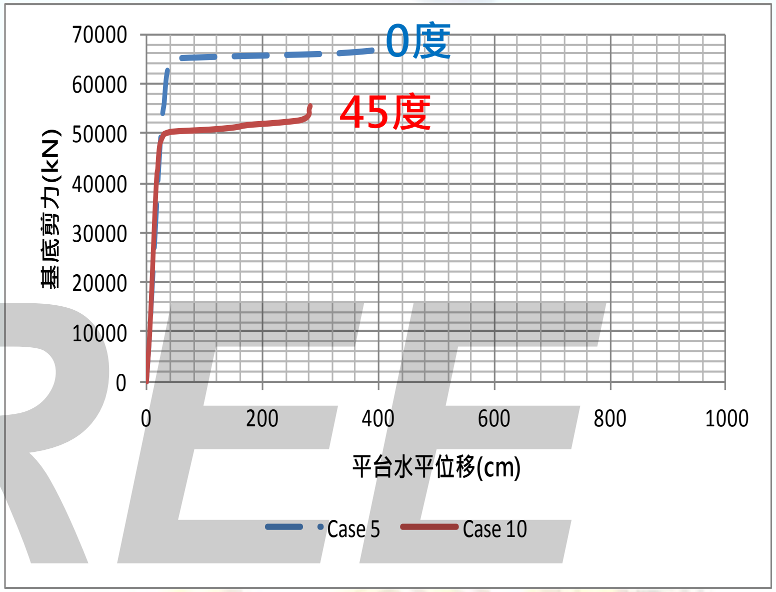
不同加載條件之韌性容量(液化土層)

設計風力、波浪與海流之加載，**對整體結構韌性無顯著影響。**

成果比較-不同束制條件之韌性容量

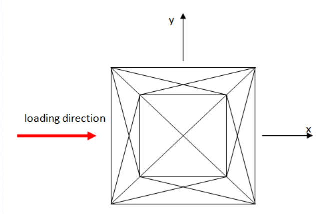


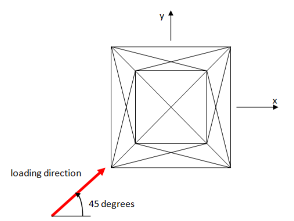
不同側推角度之韌性容量



不同側推角度之韌性容量

側推力作用於0度時，降伏與崩潰發生時之平台水平位移與基底剪力均較作用於45 度時為大，即吸收之能量較高。
但前面表列之韌性容量R 反而較低，主要由於僅以位移比代表韌性容量，而產生之差異。





耐震設計議題小結

- 在本研究之模擬情境下，套筒式風機支撐結構之**韌性容量可達6**，遠高於單樁式風機支撐結構。由於單樁式之基樁徑厚比(D/t)高，強度可能由局部挫屈控制，且無贅餘度，故單樁式不考慮結構韌性。
- **土壤液化**造成結構自然週期增加，並造成較高地震力，且**使韌性容量大幅減少**。
- 本研究案例之**第二模態對結構動態反應影響最顯著**，尤其**土壤液化時更為明顯**。
- 全世界風場大概僅台灣位於強震區，目前國際離岸風機支撐結構設計標準針對結構耐震(韌性)設計並無太多著墨，故台灣**應建立屬於本土之離岸風機支撐結構耐震設計標準**。
- 本研究推求結構韌性容量時，側推分析**未考慮風機塔架塑性**。事實上，塔柱D/t大，破壞型態可能是彈性局部挫屈，無法達到塑性。

目前耐震設計問題

- 若採475年迴歸期地震，是否允許下部結構部份區域進入塑性? <=海上結構受損很難維修
- 是否須檢核2500年迴歸期地震作用下，結構不能崩塌? <=離岸風機塔柱屬薄殼斷面，可能發生彈性局部挫屈而倒塌，恐難達到塑性。

感謝聆聽　敬請指教