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Position Paper of

FEM Product Group Cranes and Lifting Equipment - Sub-Group Tower and harbor Cranes

"Guidelines for considering tower crane loads on supporting structures"

Legal Note*: This publication is only for guidance and gives an overview regarding the assessment of tower crane loads on supporting structures. It neither claims to cover any aspect of the matter, nor does it reflect all legal aspects in detail. It is not meant to, and cannot, replace own knowledge of the pertaining directives, laws and regulations. Furthermore the specific characteristics of the individual products and the various possible applications have to be taken into account. This is why, apart from the assessments and procedures addressed in this guide, many other scenarios may apply.*

FEM

Created in 1953, the European Materials Handling Federation (www.fem-eur.com) represents, defends and promotes European manufacturers of materials handling, lifting and storage equipment including cranes and lifting equipment and particular tower cranes."

The FEM Product Group Cranes and Lifting Equipment's - Subgroup Tower and Harbor Cranes - mission is to represent the interest of tower crane manufacturers in Europe."

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0-Foreword / Introduction

This position paper reflects the opinion of FEM with regard to state of the art. It clarifies the different responsibilities regarding the design of supporting structures for the installation of tower cranes on European construction sites.

This document provides information on the interpretation and use of the static data contained in the instruction manual of tower cranes. The explanations are intended to contribute to plan safe and economical crane operations by giving guidelines for determining loads for the design of crane supporting structures.

These guidelines should be considered as minimum requirements. National regulations may require different rules and/or safety factors.

I-Tower cranes loads on supporting structures

I-1 General

Tower crane loads on supporting structures shall be determined by tower crane manufacturers using EN14439 “Cranes - Safety - Tower Cranes” which is an harmonised standard giving presumption of conformity to the EU law Machinery Directive 2006/42/EC.

This calculation differs from the Eurocodes[1] which are not fully applicable due to a different approach regarding the types of load actions and load combinations as well as the rules regarding the combination of these load actions and their partial safety factors.

I-2 Loads description

This reflects the opinion of FEM. Loads on tower crane supporting structures are necessary to enable the crane operating companies to perform all essential proof calculations. Thereby the different requirements on the loads regarding the different types of proofs have to be considered.

Loads on crane support structures are usually:

- Foundation loads of cranes erected on concrete foundation
- Corner loads of cranes using undercarriage or foundation cross
- Anchoring loads of cranes climbing in- or outside a building

These loads result from combinations of the following elementary load actions:

- Effects of gravity on the crane dead weight and on the hoist load
- Inertia loads from crane movements/accelerations of all crane drives on the crane structure and the hoist load
- Effects of the wind pressure on the crane structure and the hoist load

The load combinations, as a set of associated and dependent load actions, leading to the maximum resulting load effects of the tower crane on supporting structure are highly dependent on the tower crane configuration. Thereby the general design of the tower crane, the jib length, the tower height, the position of the rotating part

[1] Eurocode http://www.eurocode-online.de/

of the slewing tower crane related to the fixed part of the crane structure, the size and position of the hoist load, the combination of several inertia loads and/or the angle of attack of the wind are the most important parameters.

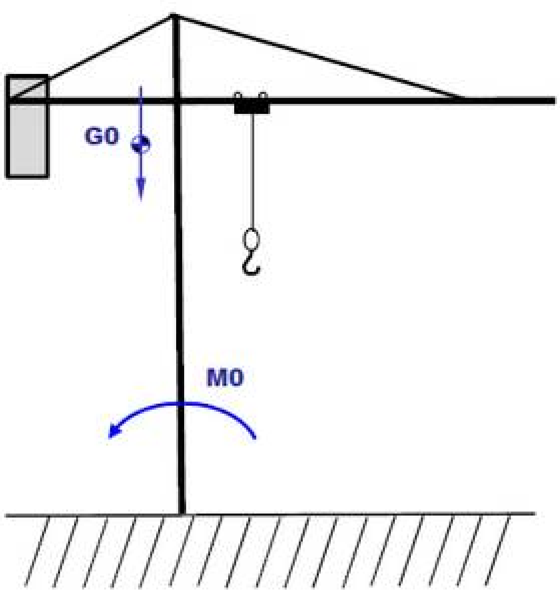
Usually the maximum loads, resulting from a large number of crane configurations, are given for each of these following situations:

- In-service loads, including maximum in-service-wind
- Out-of-service loads, with storm from rear and storm from front or alternatively storm from all sides
- Erection loads, mainly provided as foundation loads for cranes erected on concrete foundation

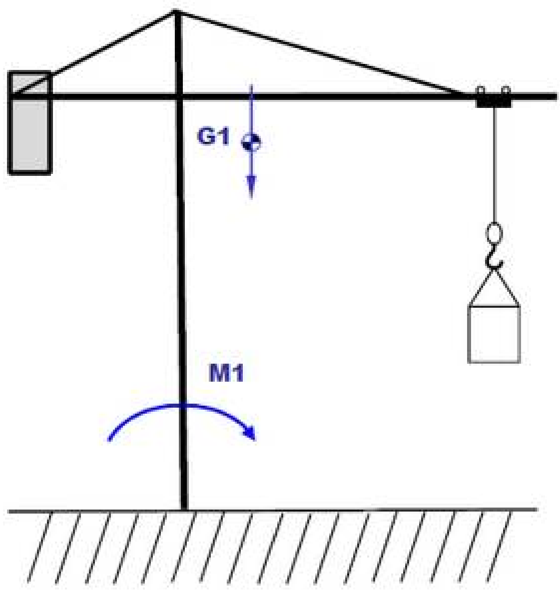
Generally loads on crane supporting structures provided by tower crane manufacturers are characteristic loads without any partial safety factors included nor dynamic factors, i.e. amplification factor due to the dynamic response of the cranes structure under the load in consideration. Likewise, second order effects, as an increase of the loading effects due to the cranes deformations, are usually not included unless clearly mentioned. A separation of these resulting loads into elementary load actions is not intended due to the above described dependency of the load action onto the crane and crane configuration.

I-3 Permanent and live loads

The reaction forces (corner reactions, foundation loads, or anchoring loads) of a tower crane predominantly consist of the resulting load moment.



Example of a resulting load moment M0
for a tower crane out-of-service



Example of a resulting load moment M1
for a tower crane in-service

The direction of the load moment is not constant as it depends at each instant on the slewing crane part position relative to the fixed part of the tower crane. This behaviour applies to in-service as well as to out-of-service situations.

It is essential to consider the variable dead weight load moment effect as a major difference to a usually invariant dead weight load effect considered for buildings. As a consequence, all loads including the dead weight moment are seen as a live load for the supporting structure except the dead weight force to be considered as a permanent effect.

II- Design of supporting structure

II-1 General

The design and proof calculation of supporting structures for tower cranes are conducted or contracted by the crane operating company usually in accordance with the Eurocodes together with national rules laid down in national annexes.

As an assumption, all tower cranes erected on construction sites are assimilated to temporary structures.

Therefore the requirements regarding fatigue or e.g. a minimum reinforcement grade of concrete foundations as required for durable buildings usually do not apply.

II-2 Loads classification

According to EN1990[2], load actions are classified as (with indication of typical partial safety factors γ):

Permanent (G)	$\gamma_G = 1{,}35$
Variable (Q)	$\gamma_Q = 1{,}5$
Accidental (A)	$\gamma_A = 1{,}0$

Note 1: These partial safety factors γ are applicable to most of ultimate limit state load combinations, refer to clause II-3.

According to EN 14439[3] the load actions and load combinations are classified as (with indication of typical partial safety factors γ):

Regular loads (A)	$\gamma_p = 1{,}34$
Occasional loads (B)	$\gamma_p = 1{,}22$
Exceptional loads (C)	$\gamma_p = 1{,}10$

Note 2: The partial safety factors γ_p indicated above give a simplified overview of the full set of partial safety factors that can slightly differ depending on individual load action under consideration.

Note 3: For standard applications of tower cranes for construction works, no accidental situations (e.g. earth quake) are considered for the foundation design.

[2] EN 1990:2002 + A1:2005 + A1:2005/AC:2010 “Eurocode – Basis of structural design“
[3] EN 14439 “Cranes - Safety - Tower Cranes”

II-3 Ultimate limit states proof (EN 1990:2002 + A1:2005 + A1:2005/AC:2010 – Clause 6.4):

The design of all crane supporting structural points is mainly governed by the ultimate limit states proof comprising:

- Stability (EQU)
- Internal failure or excessive deformation of the structure (STR)
- Failure or excessive deformation of the ground (GEO)

The general load combinations are defined as:

$$\Sigma\gamma_{G,j}G_{k,j} \text{ "+" } \gamma_{Q,1}Q_{k,1} \text{ "+" } \Sigma\gamma_{Q,i}\psi_0 Q_{k,i}$$

with

γ = *partial safety factor*

$Q_{k,1}$ = *main variable action*

$Q_{k,i}$; $i > 1$ = *accompanying variable action(s)*

ψ_0 = *factor for combination of accompanying variable action(s) with the main variable action*

The reaction forces of a tower crane on a supporting structure predominantly consist of the resulting bending moment generated by the dead weight load, the wind load and inertia loads.

As indicated in chapter I, due to the multitude of crane configurations only the most unfavourable resulting load combinations are usually provided for each typical crane situation as crane in service, crane out of service and crane during erection.

For each of these typical crane situations, the effects of all permanent and variable load actions are considered as one single variable load action on the supporting structure with their full amount. (refer also to EN 1991-3:2006/AC:2012[4] – Clause 2.2.2 (6) – Actions induced by cranes and machinery)

Due to this consideration, the load combinations can be simplified by:

$$\Sigma\gamma_{G,j}G_{k,j} \text{ "+" } \Sigma\gamma_{Q,i}Q_{k,i}$$

with

$\Sigma\gamma_{Q,i}Q_{k,i}$ = *result of all variable actions without reduction factor* ψ_0

II.3.1 Proof of strength of local supporting structures for undercarriage or cruciform base

Generally the resulting load moment from the crane on local point of supporting structures induces tensile and compressive loads on the point of contact between the tower crane and the supporting structures. Together with the rotational behaviour, all load moment effects should be multiplied uniformly with the partial safety factor for live loads (γ_Q). Only the dead weight force should be considered as a permanent effect and should be multiplied by a different partial safety factor (γ_G).

The maximum tensile and compressive load effects as well the local shear load effect result from load combinations LC1 and LC2 according to table 1.

[4] EN 1991-3:2006/AC:2012 "Eurocode 1 – Actions on structures part 3 Actions induced by cranes and machinery"

	Max compressive load LC1	Max tensile load LC2
Permanent load from the crane (dead weight vertical action)		
- Favourable	—	γ_{Ginf}
- Unfavourable	γ_{Gsup}	—
Variable load from the crane (including weight moment action)		
- Favourable	—	—
- Unfavourable	γ_{Qsup}	γ_{Qsup}

Table 1: Load combinations for tower crane local actions on supporting structures

However for tower cranes on undercarriage or cruciform base no permanent corner loads can be defined due to the rotation of the crane and the possibility of lifting at each corner. In this case the local load effect (corner reaction) has to be multiplied with the factors for live loads (γ_Q) entirely.

Due to the nature of the different load combinations acting on a tower crane, it is necessary to consider different values for partial safety factors depending on the probability of occurrence of the load combination in accordance with the classification given in tower crane design standards.

Crane loads provided without any dynamic factor nor second order effect (that can affect resulting loads for "high cranes"), should be increased by a general amplification factor ϕ. When this factor is not indicated by the crane manufacturer, a minimum value of 1,10 should be used for factor ϕ (only applicable for the superior partial safety factor).

Simplified table of partial safety factors proposed for the design of local supporting structures of undercarriage or cruciform base:

	Partial safety factors if second order and dynamic effects are included by the crane manufacturer [1)]			Partial safety factors if second order and dynamic effects are not included by the crane manufacturer (ϕ = 1,10)		
Load case	γ_{Gsup}	γ_{Ginf}	γ_{Qsup}	γ_{Gsup}	γ_{Ginf}	γ_{Qsup}
In service (in operation)	Not applicable for tower cranes on undercarriage or cruciform base		1,35	Not applicable for tower cranes on undercarriage or cruciform base		1,50
Out of service (storm from rear)			1,22			1,35
Out of service (storm from front/side)			1,10			1,22
In erection (assembly)			1,22			1,35

Table 2: Partial safety factors for cranes on undercarriage or cruciform base

1) These partial safety factors are related to the actual crane design standards EN 14439[5] and EN 13001[6], amplified by a factor of 1,1 to consider the situation of construction site.

For cranes on chassis, a proof of stability is not required as the crane stability checked by the crane manufacturer already defines the ballast required on the chassis according to crane standard.

II.3.2 Proof of strength of concrete block foundation

For tower cranes on concrete block foundation, it is also necessary to check the concrete block strength, in addition to the local effect of each tower crane structure point of connection to the supporting structure.

Crane loads provided without any dynamic factor nor second order effect (that can affect resulting loads for "high cranes"), can be increased by a general amplification factor ϕ. In case this factor is not indicated by the crane manufacturer, a minimum value of 1,10 should be used for factor ϕ (only applicable for the superior partial safety factor).

Table of partial safety factors proposed for the design of concrete block foundation with embedded local supporting structures (e.g. foundation anchors):

	Partial safety factors if second order and dynamic effects are included by the crane manufacturer 1)			Partial safety factors if second order and dynamic effects are not included by the crane manufacturer (ϕ = 1,10)		
Load case	γ_{Gsup}	γ_{Ginf}	γ_{Qsup}	γ_{Gsup} 2)	γ_{Ginf}	γ_{Qsup}
In service (in operation)	1,35	1,0	1,35	1,35	1,0	1,50
Out of service (storm from rear)	1,22	1,0	1,22	1,22	1,0	1,35
Out of service (Storm from front/side)	1,10	1,0	1,10	1,10	1,0	1,22
In erection (assembly)	1,22	1,0	1,22	1,22	1,0	1,35

Table 3: Partial safety factors for cranes on concrete block foundation

1) These partial safety factors are related to the actual crane design standards EN 14439[7] and EN 13001[8], amplified by a factor of 1,1 to consider the situation of construction site.

2) Amplification factor ϕ has not to be considered for dead weight force on foundation due to no increase with theory second order.

This check shall be consistent with the load combinations considered for the check of each local effect on supporting structures.

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5 EN14439 "Cranes - Safety - Tower Cranes"
6 EN 13001 "Cranes – General design"
7 EN14439 "Cranes - Safety - Tower Cranes"
8 EN 13001 "Cranes – General design"

II.3.3 Proof of stability of concrete block foundation

The global stability under ultimate limit state shall be checked considering the appropriate criteria depending on the concrete block foundation configuration in the ground. As a general guideline for shallow foundation, the resulting load eccentricity shall not be greater than 1/2 of the concrete block outside dimension in any direction, when considering the different load combinations used for the proof of strength.

Note 1: This guideline is defined considering the temporary installation of the tower crane on the construction site. In case of special application (e.g. permanent installation on a stock yard) a maximum eccentricity of 1/3rd may be necessary.

Note 2: In general, the proof of global stability checked for the serviceability limit state will govern the stability proof.

II-4- Serviceability limit states proof (EN 1990:2002 + A1:2005 + A1:2005/AC:2010[9] – Clause 6.5)

The serviceability limit state proof for tower crane supporting structures mainly consists in checking the risk of ground deformation and global stability.

The load combinations are simplified by:

$$\Sigma G_{k,j} \text{ "+" } Q_{k,1} \text{ "+" } \Sigma \psi_0 Q_{k,i}$$

For the same reason as explained in II-3, the load combinations can then be simplified by:

$$\Sigma G_{k,j} \text{ "+" } \Sigma Q_{k,i}$$

with

$\Sigma Q_{k,i}$ = *result of all variable actions without reduction factor*

As a general guideline for shallow foundation, the resulting load eccentricity shall not be greater than 1/3rd of the concrete block outside dimension in any direction.

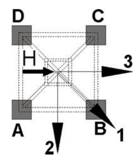
Note 1: This guideline is defined considering the temporary installation of the tower crane on the construction site. In case of special application (e.g. permanent installation on a stock yard), a maximum eccentricity of 1/6th for the load combination “crane in service”, assumed as quasi-permanent ELS combination, may be necessary.

Note 2: For cranes on chassis, a proof of stability is not required as the crane stability checked by the crane manufacturer already defines the ballast required on the chassis according to crane standard.

[9] EN 1990:2002 + A1:2005 + A1:2005/AC:2010 “Eurocode – basis of structural design”

Example 1: Crane on undercarriage or cruciform base (typical tower crane manufacturer data without theory second order effects)

Corner reactions (characteristic load values, numerical example)

Crane type:												Jib:
Tower system:								tower selection length:				
Base tower:												
Crane base:										Track: 6 m Wheel gauge: 6 m		
No. of tower sections	Hook height [m]	Central ballast [to]	Corner pressure in operation [kN], M_D = 325 kNm					Corner pressure out of operation [kN], M_D = 0 kNm				
				Position of jib			Hor. force		Position of jib			Hor. force
			Corner	1	2	3	[kN]	Corner	1	2	3	[kN]
5	40,30	86,110	A	442	607	242	56	A	331	737	89	196
			B	**730**	676	641		B	**991**	737	737	
			C	442	277	641		C	331	89	737	
			D	154	208	242		D	0	89	89	

Proposal of partial safety factors for corner reactions acc. to 1st order theory

	Safety factors if second order and dynamic effects are not included by the crane manufacturer (ϕ = 1,10)		
Load case	γ_{Gsup}	γ_{Ginf}	γ_{Qsup}
In service (in operation)	Not applicable for cranes on undercarriage or cruciform base		1,5
In erection (assembly)			1,35
Out of service (storm from rear)			1,35
Out of service (Storm from front/side)			1,22

See clause II.3.1 - table 2

If out of operation (out of service) is not clearly divided in storm from rear or storm from front/side the safety factor for storm from rear has to be taken into account (save assumption).

Max. corner reactions (design load values, numerical example)

No. of tower sections	Hook height [m]	Central ballast [to]	Design Corner pressures in operation [kN]					Design Corner pressures out of operation [kN]				
				Position of jib			Hor. force		Position of jib			Hor. force
			Corner	1	2	3	[kN]	Corner	1	2	3	[kN]
5	40,30	86,110	max	**1095**	1014	962	36	max	**1338**	995	995	66

Note 1: The above given global characteristic horizontal load and torque moment is recalculated into a max. local horizontal design shear force considering 4 corners acting equally.

Determination of horizontal force:

In operation (in service):

$$H_d = \gamma_{Q,\sup} * \sqrt{\left(\frac{M_D}{4 * a/\sqrt{2}}\right)^2 + \left(\frac{H_k}{4}\right)^2}$$

with a = system width of undercarriage

Out of operation (out of service):

$$H_d = \gamma_{Q,\sup} * \frac{H_k}{4}$$

Attention shall be given to the effective boundary conditions attached to each of the 4 corners (e.g. no horizontal fixation of all corners or for a mobile configuration on rail). Other transmission models may be used accordingly, taking into account the prevailing bearing conditions (e.g. sliding capability of corners or H-force application perpendicular to the rail).

Note 2: As no permanent corner load can be calculated it will be set to 0. Thus the corner reaction is seen as an entire live load and is multiplied with the corresponding partial safety factor γ_Q.

Example 2: Crane on concrete foundation block (typical tower crane manufacturer data without theory second order effects)

EXPLANATION AND LEGENDS OF THE CONCRETE BLOCK CALCULATION TABLES

> *i*
> The indicated reactions do not include a coefficient for dead weights nor a dynamic coefficient for the lifting load.
> It is appropriate to take into consideration the usual or standardized safety coefficients valid for the indicated loads and reactions.
> As usual, a negative (–) reaction corresponds to a pressure on a bearing and a positive (+) reaction to a traction.

MV = Wind moment

MR = Moment resulting from dead weight + load + centrifugal force

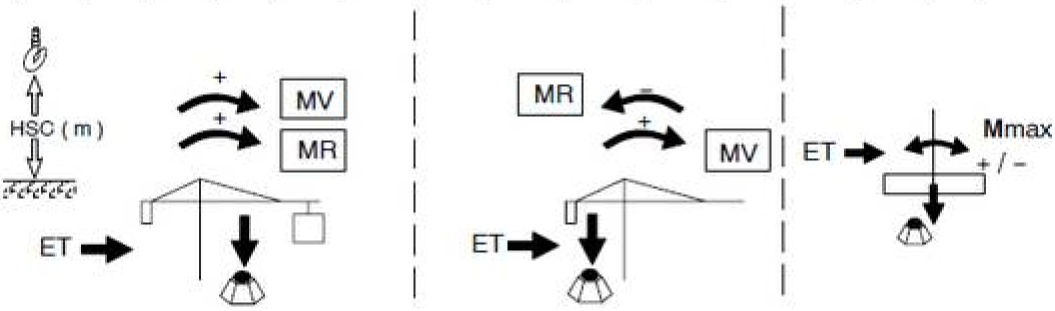
ET – Shearing force except the mast torsional moment

Mmax = Maximum moment

Cmax = Max. torsional moment on the masts

Cmax : xxxx kNm

	●					■					During erection		
HSC	MV	MR	Mmax		ET	MV	MR	Mmax		ET	Mmax		ET
(m)	(kNm)	(kNm)	(kNm)	(kN)	(kN)	(kNm)	(kNm)	(kNm)	(kN)	(kN)	(kNm)	(kN)	(kN)



Case of rear wind

Maximum moment = **MV** + **MR**

Or

Case of perpendicular wind

Maximum moment = $\sqrt{(\mathbf{MV}^2 + \mathbf{MR}^2)}$

This is the most unfavourable case which appears in the table.

The maximum moment depends on the values of the dead weight moment (**MR**) and the wind moment (**MV**):

If **MV** < 2 x **MR** then max. moment = **MR**

If **MV** > 2 x **MR** then max. moment = **MV**– **MR**

Maximum shearing force on fixing angle

$$\frac{ET}{2} + \frac{Cmax}{d}$$

d = mast width

Max. shearing force on fixing angle $= \frac{ET}{2}$

In out of service condition, the crane is in weathervaning position. **C**max = 0

Cmax = 270. kNm

	●					■					En montage		
HSC (m)	MV (kNm)	MR (kNm)	Mmax (kNm)	(kN)	ET (kN)	MV (kNm)	MR (kNm)	Mmax (kNm)	(kN)	ET (kN)	Mmax (kNm)	(kN)	ET (kN)
40.2	1843.	978.	2086.	885.	57.	4763.	-1159.	3605.	765.	193.	-2175.	474.	31.
41.8	1929.	978.	2163.	891.	57.	5058.	-1159.	3899.	771.	198.	-2214.	479.	31.
43.5	2026.	979.	2250.	905.	58.	5575.	-1159.	4416.	785.	208.	-2253.	494.	32.
45.2	2115.	980.	2331.	910.	59.	5894.	-1159.	4735.	790.	213.	-2292.	499.	33.
46.8	2205.	981.	2413.	916.	59.	6244.	-1159.	5085.	796.	219.	-2331.	505.	33.
48.5	2307.	982.	2507.	930.	60.	6674.	-1159.	5515.	810.	226.	-2370.	519.	34.
50.2	2400.	983.	2593.	938.	61.	7041.	-1159.	5882.	818.	232.	-2409.	527.	35.
51.8	2493.	984.	2680.	944.	62.	7412.	-1159.	6253.	824.	237.	-2448.	533.	35.
53.5	2600.	985.	2780.	965.	62.	7876.	-1159.	6717.	845.	244.	-2487.	554.	36.
55.2	2697.	985.	2871.	971.	63.	8268.	-1159.	7109.	851.	250.	-2526.	560.	37.
56.8	2795.	986.	2963.	976.	64.	8665.	-1159.	7506.	856.	255.	-2565.	565.	38.

Cmax = 270. kNm

Vent 100km/h

HSC (m)	Mmax (kNm)	(kN)	ET (kN)
16.8	1763.	643.	97.
18.5	1853.	658.	99.
20.2	1943.	663.	101.
21.8	2038.	669.	103.
23.5	2143.	683.	106.
25.2	2246.	689.	108.
26.8	2353.	694.	110.
28.5	2471.	709.	112.
30.2	2585.	714.	114.
31.8	2702.	720.	116.
33.5	2832.	734.	119.
35.2	2956.	740.	121.
36.8	3082.	745.	123.
38.5	3223.	760.	126.
40.2	3355.	765.	128.
41.8	3489.	771.	130.
43.5	3640.	785.	133.
45.2	3780.	790.	135.
46.8	3923.	796.	137.
[illegible]	[illegible]	[illegible]	[illegible]

System of tower: D [m] = 1,84		CRANE IN SERVICE			CRANE OUT OF SERVICE Storm from rear			CRANE OUT OF SERVICE Storm from side/front			CRANE DURING ERECTION		
Foundation loading without theory second order. Partial safety factors according table 3 clause II.3.2 with ϕ = 1,10 should be used	Cmax	C_k	270	kNm	C_k	0	kNm	C_k	0	kNm	C_k	270	kNm
	Weight (W)	V_k	885	kN	V_k	765	kN	V_k	765	kN	V_k	474	kN
	Moment (Mmax)	M_k	2086	kNm	M_k	3605	kNm	M_k	3355	kNm	M_k	2175	kNm
	Shear (Tmax)	H_k	57	kN	H_k	193	kN	H_k	128	kN	H_k	31	kN
Ultimate Loads													
Local Strength (per corner)		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q	
max. resulting compressive loads [kN]	see formula [1]	1,35	1,50	**-1501**	1,22	1,35	**-2104**	1,10	1,22	**-1783**	1,22	1,35	**-1273**
max. resulting tension loads [kN]	see formula [2]	1,00	1,50	**981**	1,00	1,35	**1679**	1,00	1,22	**1382**	1,00	1,35	**1010**
max. resulting shear load [kN]	see formula [3]	--	1,50	**98**	--	1,35	**130**	--	1,22	**78**	--	1,35	**70**
Global Strength		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q	
vertical load [kN]	γ_{Gsup}*V_k	1,35	--	**1195**	1,22	--	**933**	1,10	--	**842**	1,22	--	**578**
overturning moment [kNm]	γ_{Qsup}*M_k	--	1,50	**3129**	--	1,35	**4867**	--	1,22	**4093**	--	1,35	**2936**
horizontal load [kN]	γ_{Qsup}*H_k	--	1,50	**86**	--	1,35	**261**	--	1,22	**156**	--	1,35	**42**
slewing moment [kNm]	γ_{Qsup}*C_k	--	1,50	**405**	--	1,35	**0**	--	1,22	**0**	--	1,35	**365**
Global Stability		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q	
vertical load [kN]	γ_{Ginf}*V_k	1,00	--	**885**	1,00	--	**765**	1,00	--	**765**	1,00	--	**474**
overturning moment [kNm]	γ_{Qsup}*M_k	--	1,50	**3129**	--	1,35	**4867**	--	1,22	**4093**	--	1,35	**2936**
horizontal load [kN]	γ_{Qsup}*H_k	--	1,50	**86**	--	1,35	**261**	--	1,22	**156**	--	1,35	**42**
slewing moment [kNm]	γ_{Qsup}*C_k	--	1,50	**405**	--	1,35	**0**	--	1,22	**0**	--	1,35	**365**
Serviceability Loads													
Global Stability		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q		γ_G	γ_Q	
vertical load [kN]	γ_G*V_k	1,00	--	**885**	1,00	--	**765**	1,00	--	**765**	1,00	--	**474**
overturning moment [kNm]	γ_Q*M_k	--	1,00	**2086**	--	1,00	**3605**	--	1,00	**3355**	--	1,00	**2175**
horizontal load [kN]	γ_Q*H_k	--	1,00	**57**	--	1,00	**193**	--	1,00	**128**	--	1,00	**31**
slewing moment [kNm]	γ_Q*C_k	--	1,00	**270**	--	1,00	**0**	--	1,00	**0**	--	1,00	**270**

[1] $$F_{c,d} = -\gamma_{Q\,sup}\cdot\frac{M_k}{D\cdot\sqrt{2}} - \gamma_{G\,sup}\cdot\frac{V_k}{4}$$

[2] $$F_{t,d} = +\gamma_{Q\,sup}\cdot\frac{M_k}{D\cdot\sqrt{2}} - \gamma_{G\,inf}\cdot\frac{V_k}{4}$$

[3] $$S_{d,\max} = \gamma_{Q\,sup}\cdot\left(\frac{H_k}{2}+\frac{C_k}{D}\right)$$

Note : as a conservative approach, formula [3] is proposed considering here only 2 feet acting for the horizontal load