Tower Cranes & Foundations
The Interface & CIRIA C654

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EXAMPLES OF TOWER CRANE FOUNDATION TYPES
Rail mounted
Piled Base
Piled Base
Grillage Base
Grillage Base
SELECTION OF FOUNDATION TYPE

This will depend on:

The class of crane – Light, Medium or Heavy duty

and

The ground conditions – Very soft clay to Rock

and

The site constraints – open area or congested inner city
The Interface

Mechanical

‘Thou’ (μm)

EN 13001-02
Regular, Variable, & Occasional Loads

Civil

1/16 (mm)

EN1990
Permanent, Quasi-Permanent, Variable, & Accidental Actions
Foundation designs are currently carried out in accordance with CIRIA C654 Tower Crane Stability.

This guide published in 2006 anticipated that the information from crane owners would in future be more detailed so as to align with Eurocodes.
CIRIA C654 Tower Crane Stability is currently being re-written to align with Eurocodes.

This is proving challenging due to the misalignment of the product design code with the general Eurocodes, and the different information provided by different manufacturers.
Typical Foundation Loads

<table>
<thead>
<tr>
<th>In Operation</th>
<th>Out of Operation</th>
<th>Erection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Storm from rear</td>
<td>Storm from front</td>
</tr>
<tr>
<td>M (kNm)</td>
<td>H (kN)</td>
<td>V (kN)</td>
</tr>
<tr>
<td>3343</td>
<td>65</td>
<td>939</td>
</tr>
</tbody>
</table>

3488  29  581
Draft revision to C654 treatment of the above loads

The Self Weight of the tower crane and of the foundation is taken as a Permanent Action

All other loads are taken as Variable Actions
Design of a simple pad base foundation

There are 3 main aspects to the design

a) Stability – the EQU limit state

b) Geotechnical Capacity – the GEO limit states

c) Structural Design – STR limit state
Example Design Method

Provisional – Still Under Development

Gravity Crane Base
In order to illustrate the above we will use loading data from the Liebherr 280 EC-H 12 Litronic at a hook height of 47.9m with a 75m jib

Ground conditions will be taken as a cohesive material with shear strength of 200 kN/m²
The EQU limit state

Erection Case

Wt of base = 6.5m x 6.5m x 1.4m x 24 kN/m³ = 1420 kN

Wt of crane = 581 kN

Total = 2001 kN

Stabilising Moment = 2001 kN x 6.5m / 2 x 0.9γ = 5852 kNm

Destabilising Moment = (3488 + 29 kN x 1.4 m) x 1.5γ = 5292 kNm

Stabilising > Destabilising - OK
Storm Case

Wt of base = 6.5m x 6.5m x 1.4m x 24 kN/m³ = 1420 kN

Wt of crane = 912 kN

Total = 2132 kN

Stabilising Moment = 2332 kN x 6.5m / 2 x 1.0γ = 7579 kNm

Destabilising Moment =(4270 + 87 kN x 1.4 m)x1.0γ = 4391 kNm

Stabilising > Destabilising - OK
The GEO limit states

There are 2 Ultimate GEO limit states to check, one with a material factor of 1.0 on the soil properties, and the other with a capacity reduction factor – in this case 1/1.4 on the soil strength.

The maximum soil pressures occur with the jib at an angle to the base. Part of the base may not be in contact with the ground.
Contact area

Note that the ground capacity varies with the loaded shape
The pressure is calculated based on Meyerhof for an equivalent uniform pressure distribution over a reduced rectangular area.
GEO limit state ULS Combination 1

Bearing capacity – there are 2 cases to check

Factor the variable load (moment) by 1.5

Factor the permanent load (Base and Crane wt.) by Case 1 Case 2

Calculate the area of ground under load for a variety of jib angles for each case.

Calculate the bearing pressure on the ground

Calculate the bearing capacity of the ground for each pressure and loaded area

Check that Capacity > Applied Load
GEO ULS Combination 1 Case 1

Erection

Stabilising Action = 2001 kN \times 1.35\gamma = 2701 kN

Destabilising Moment = (3488 + 29 kN \times 1.4 \text{ m}) \times 1.5\gamma = 5292\text{kNm}

Eccentricity = \frac{5292\text{kNm}}{2701\text{kN}} = 1.95\text{m}

Width of soil loaded = 6.5m - 2 \times 1.95m = 2.6m

Soil Capacity = A' (c_{ud} N_c b_c s_c i_c + q)

Soil Capacity = 9718\text{kN}

9718\text{kN} > 2701\text{ OK}
GEO ULS Combination 1 Case 2

Erection

Stabilising Action = 2001 kN x 1.0γ = 2001 kN

Destabilising Moment = (3488 + 29 kN x 1.4 m) x 1.5γ = 5292 kNm

Eccentricity = 5292 kNm / 2001 kN = 2.64 m

Width of soil loaded = 6.5 m – 2 x 2.64 m = 1.22 m

Soil Capacity = \( A' \left( c_{ud} N_c b_c s_c i_c + q' \right) \)

Soil Capacity = 4350 kN

4350 kN > 2001 OK
Sliding

The horizontal load is a variable load and hence factored by 1.5

The soil resistance is unfactored, but the friction factor between the concrete and soil needs to be incorporated. EC7 does not give any guidance, but BS8002 suggests 0.75

Horizontal Action = \(29 \times 1.5\gamma = 43.5 \text{kN}\)

Resistance = \(100 \text{kN/m}^2 \times 1.22\text{m} \times 6.5\text{m} \times 1.0\gamma \times 0.75 = 594 \text{kN}\)
GEO limit state ULS Combination 2

Bearing Capacity

Factor the variable load (moment) by 1.3

Factor the permanent load (Base and Crane wt.) by 1.0

Calculate the area of ground under load for a variety of jib angles for each case.

Calculate the bearing capacity of the ground for each pressure and loaded area

Compare this with the failure capacity of the ground with strength reduced by 1.4
GEO ULS Combination 2

Erection

Stabilising Action = 2001 kN x 1.0γ = 2001 kN

Destabilising Moment = (3488 + 29 kN x 1.4 m) x 1.3γ = 4587 kNm

Eccentricity = 4587 kNm / 2001 kN = 2.29 m

Width of soil loaded = 6.5 m - 2 x 2.29 m = 1.92 m

Soil Capacity = A' (\(c_{ud} N_c b_c s_c i_c + q'\))

Soil Capacity = 8221 kN

8221 kN > 2001 kN  OK
**Sliding**

The horizontal load is a variable load and hence factored by 1.3

The soil resistance is factored by 1/1.4, and the friction factor between the concrete and soil is incorporated.

\[ \text{Horizontal Action} = 29 \times 1.3\gamma = 37.7 \text{ kN} \]

\[ \text{Resistance} = 100 \text{ kN/m}^2 \times 1.92\text{m} \times 6.5\text{m} \times 0.75 / 1.4\gamma = 668 \text{ kN} \]
GEO limit state SLS

Calculate the settlement of the ground under SLS loads and confirm this is acceptable with the Tower crane Manufacturer

OR

Based on UK custom and practice, calculate the bearing pressure on the ground under SLS loading, and if this is < 1/3 of the failure capacity, deem that settlements will be acceptable
STR limit state

*Design with jib orthogonal*

Take the worst case from the GEO analysis

Calculate the maximum moment which is at the point of zero shear
GEO ULS Combination 1 Case 2

Design the reinforcement
The base projects 2m beyond the tower crane leg (point of zero shear)

Ground Pressure = 2001 kN / 1.22m / 6.5 m = 252 kPa

Design moment =
252 kPa * 1.22m *(3.25 m – 1.22m/2) – 33.6kPa *(2.25m)^2/2 = 520 kNm/m
Using 25/30 concrete $f_{ck} = 25$ N/mm$^2$

Effective depth = 1.4m – 50mm cover – 40mm bar allowance = 1310mm

$K = \frac{M_{ed}}{(bd^2f_{ck})} = \frac{520 \times 10^6}{1000/1310^2 / 25} = 0.012$

Lever arm $Z = d(0.5 + \sqrt{0.25 - K / 0.9})$ but < 0.95 x d

$Z = 0.95 \times 1310 = 1245$mm

Area of reinforcement required

$A_s = \frac{M}{f_{yd} z} = \frac{520 \text{ kNm}}{(500/1.15\gamma \times 1245\text{mm})} = 960 \text{ mm}^2 / \text{m}$
Check minimum reinforcement $= 0.26 \times \left( \frac{f_{ctm}}{f_{yk}} \right) b_t d > 0.0013 b_t d$

where $f_{ctm} = 0.30 f_{ck}^{0.666} = 0.30 \times 25^{0.666} = 2.56$ Mpa

Minimum reinforcement $= 0.26 \times \left( \frac{2.56}{500} \right) \times 1000 \times 1310 \geq 0.0013 \times 1000 \times 1310$

$1744 \text{ mm}^2 / \text{ m}$ but $> 1703 \text{ mm}^2 / \text{ m}$

Hence minimum reinforcement governs $- 1744 > 960 \text{ mm}^2 / \text{ m}$
Check Shear
Design Shear at $d$ from support

$252 \text{ kPa} \times 0.94\text{m} - 33.6\text{kPa} \times 0.94\text{m} = 205 \text{ kN/m}$
Shear stress $\nu_{\text{Ed}} = \frac{205 \text{ kN/m}}{1310\text{mm}} / 1\text{m} = 0.16 \text{ kPa}$

$\nu_{\text{Rd,c}} = \frac{0.18}{\gamma_c}k(100r_l f_{\text{ck}})^{0.333} \geq 0.035k^{1.5}f_{\text{ck}}^{0.5}$

where

$\gamma_c = 1.5$
$k = 1 + (200/d)^{0.5} \leq 2.0: k = 1 + (200/1310)^{0.5} = 1.39$
$r_l = A_{\text{sl}}/bd = \frac{1744}{(1000 \times 1310)} = 0.00133$
$f_{\text{ck}} = 25 \text{ MPa}$

$\nu_{\text{Rd,c}} = \frac{0.18}{1.5} \times 1.39 \times (100 \times 0.00133 \times 25)^{0.333} \geq 0.035 \times 1.38^{1.5} \times 25^{0.5}$

$= 0.284 > 0.249 \geq 0.16 \text{ MPa}$
Pull out/push through of the anchors

The CIRIA guide states “If the manufacturer’s recommendations regarding shear reinforcement are followed, punching and pull out shear should be satisfactory”

I have yet to see any manufacturer’s recommendations regarding shear reinforcement, apart from sketches indicating where it should go.
Foundation Anchors
This is fundamentally a punching shear issue

With some types of anchor it is clear where the failure cone will occur
With others it is less clear, but Liebherr now suggest
Discussion Points

Storm from front condition – should this be a general design case or a special case?

Why can we not have loadings which are Eurocode compliant?

What load factors are appropriate to the erection case?

Are current expendable anchor designs sustainable and what can be done to improve them?