Tower Cranes & Foundations
The Interface & CIRIA C654

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EXAMPLES OF TOWER CRANE FOUNDATION TYPES

Rail mounted

Created and organised by

Pad Base

Piled Base

Piled Base

Grillage Base

Grillage Base

Wentworth House Partnership

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	Civil
‘Thou’ (µm)	1/16 (mm)
EN 13001-02 Regular, Variable, & Occasional Loads	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

In Operation			Out of Operation						Erection		
			Storm from rear			Storm from front					
M (kNm)	H (kN)	V (kN)	M (kNm)	H (kN)	V (kN)	M (kNm)	H (kN)	V (kN)	M (kNm)	H (kN)	V (kN)
3343	65	939	2836	129	910	4270	87	912	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/m^2

The EQU limit state

Erection Case

Wt of base = 6.5m x 6.5m x 1.4m x 24 kN/m^3 = 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)x1.5γ = 5292kNm

Stabilising > Destabilising - OK

Storm Case

Wt of base = 6.5m x 6.5m x 1.4m x 24 kN/m^3 = 1420 kN

Wt of crane = 912 kN

Likely to be revised

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 = 4391kNm

Stabilising > Destabilising - OK

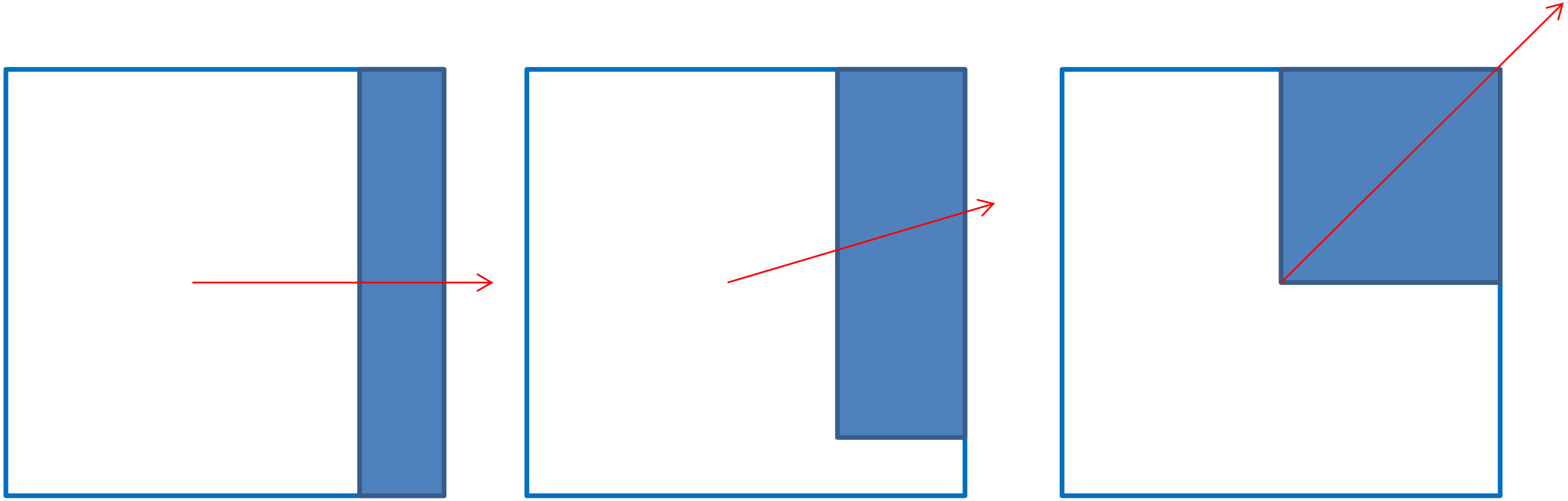
The GEO limit states

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.

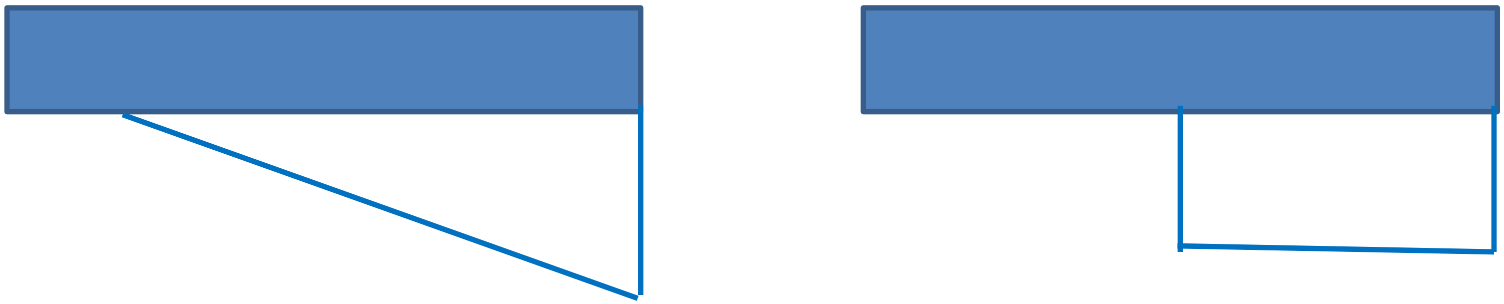
The maximum soil pressures occur with the jib at an angle to the base. Part of the base may not be in contact with the ground.

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
	1.35	1.0

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 x1.35γ = 2701 kN

Destabilising Moment =(3488 + 29 kN x 1.4 m)x1.5γ = 5292kNm

Eccentricity = 5292kNm / 2701kN = 1.95m

Width of soil loaded = 6.5m – 2 x 1.95m = 2.6m

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

Soil Capacity = 9718 kN

9718 kN > 2701 OK

GEO ULS Combination 1 Case 2

Erection

Stabilising Action = 2001 kN x1.0γ = 2001 kN

Destabilising Moment =(3488 + 29 kN x 1.4 m)x1.5γ = 5292kNm

Eccentricity = 5292kNm / 2001kN = 2.64m

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

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

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 x 1.5γ = 43.5 kN

Resistance = 100 kN/m^2 x 1.22m x 6.5m * 1.0γ * 0.75 = 594 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 x1.0γ = 2001 kN

Destabilising Moment =(3488 + 29 kN x 1.4 m)x1.3γ = 4587kNm

Eccentricity = 4587kNm / 2001kN = 2.29m

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

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

Soil Capacity = 8221 kN

8221 kN > 2001kN 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.

Horizontal Action= 29 x 1.3γ = 37.7 kN

Resistance = 100 kN/m^2 x 1.92m x 6.5m * 0.75 / 1.4γ = 668 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

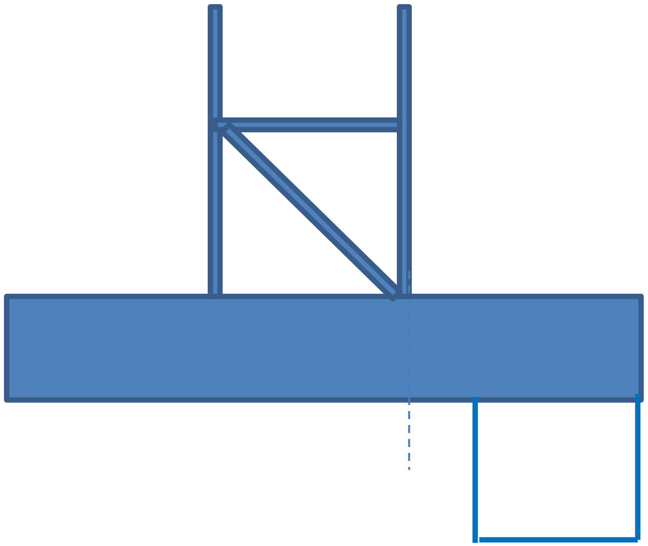
Based on UK custom and practice, calculate the bearing pressure on the ground under SLS loading, and if this is < 1/3 of the failure capacity, deem that settlements will be acceptable

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 =

$252 \text{ kPa} * 1.22\text{m} *(3.25 \text{ m} – 1.22\text{m}/2) – 33.6\text{kPa} *(2.25\text{m})^2/2 =$

520 kNm/m

Using 25/30 concrete f_{ck} = 25 N/mm^2
Effective depth = 1.4m – 50mm cover – 40mm bar allowance = 1310mm
K = M_{ed} / (bd^2f_{ck}) = 520 x 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 x 1310 = 1245mm

Area of reinforcement required
A_s = M / f_{yd} z = 520 kNm / (500/1.15γ x1245mm) = 960 mm^2 /m

Check minimum reinforcement = $0.26 \times (f_{ctm}/f_{yk})b_t d > 0.0013 b_t d$
where $f_{ctm} = 0.30 f_{ck}^{0.666} = 0.30 \times 25^{0.666} = 2.56$ Mpa

Minimum reinforcement = $0.26 \times (2.56/500) \times 1000 \times 1310 \geq 0.0013 \times 1000 \times 1310$
$1744\ mm^2 / m$ but $> 1703\ mm^2 / m$

Hence minimum reinforcement governs – $1744 > 960\ mm^2 / m$

Check Shear

Design Shear at d from support

252 kPa * 0.94m - 33.6kPa *0.94m = 205 kN/m

Shear stress v_{Ed} = 205 kN/m / 1310mm / 1m = 0.16 kPa

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

where

$\gamma_c = 1.5$

$k = 1 + (200/d)^{0.5} \leq 2.0$: $k = 1 + (200/1310)^{0.5} = 1.39$

$r_l = A_{sl}/bd = 1744/(1000\text{x } 1310) = 0.00133$

$f_{ck} = 25$ MPa

$v_{Rd,c} = (0.18/1.5)\text{x } 1.39 \text{ x } (100 \text{ x } 0.00133 \text{ x } 25)^{0.333} \geq 0.035 \text{ x } 1.38^{1.5}\text{x } 25^{0.5}$

= 0.284> 0.249 ≥ 0.16 MPa

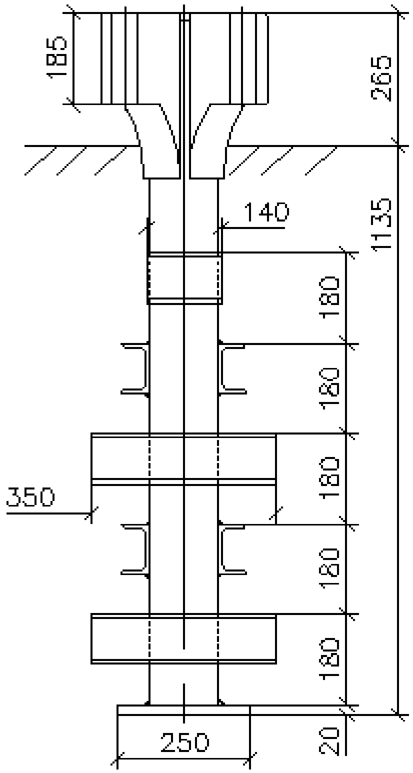
The exciting bit

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”

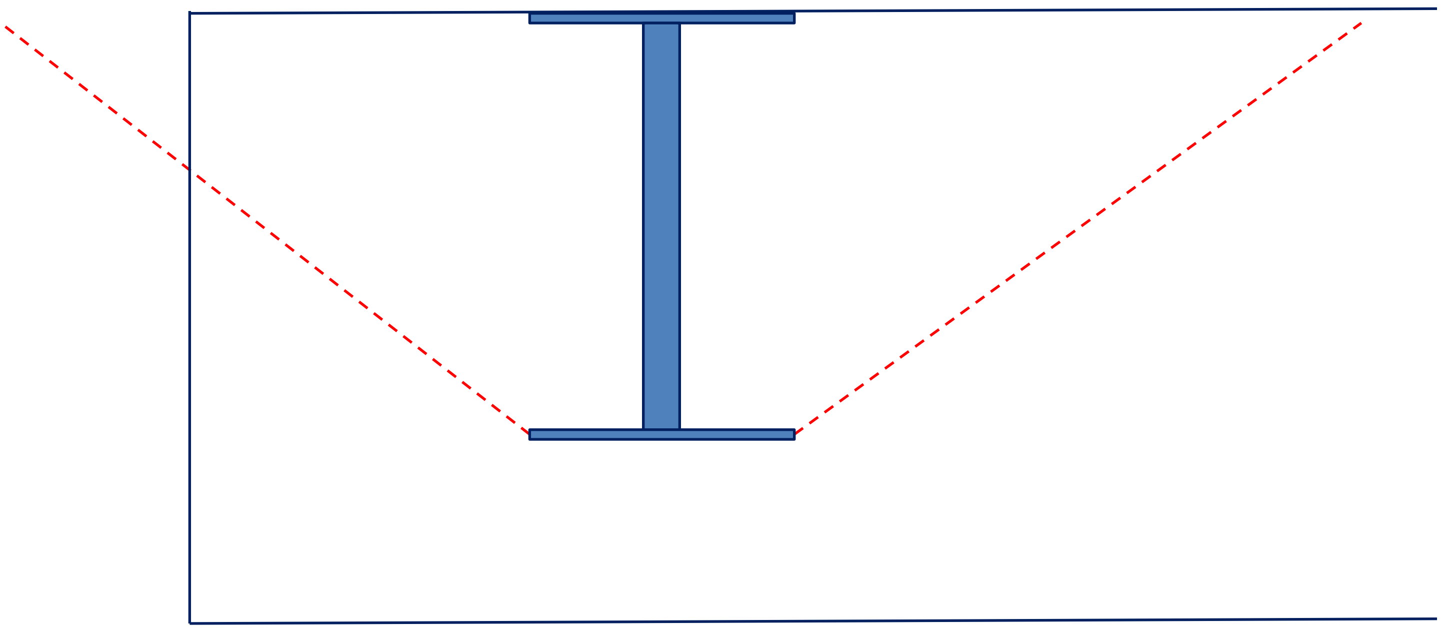
I have yet to see any manufacturer’s recommendations regarding shear reinforcement, apart from sketches indicating where it should go.

Foundation Anchors

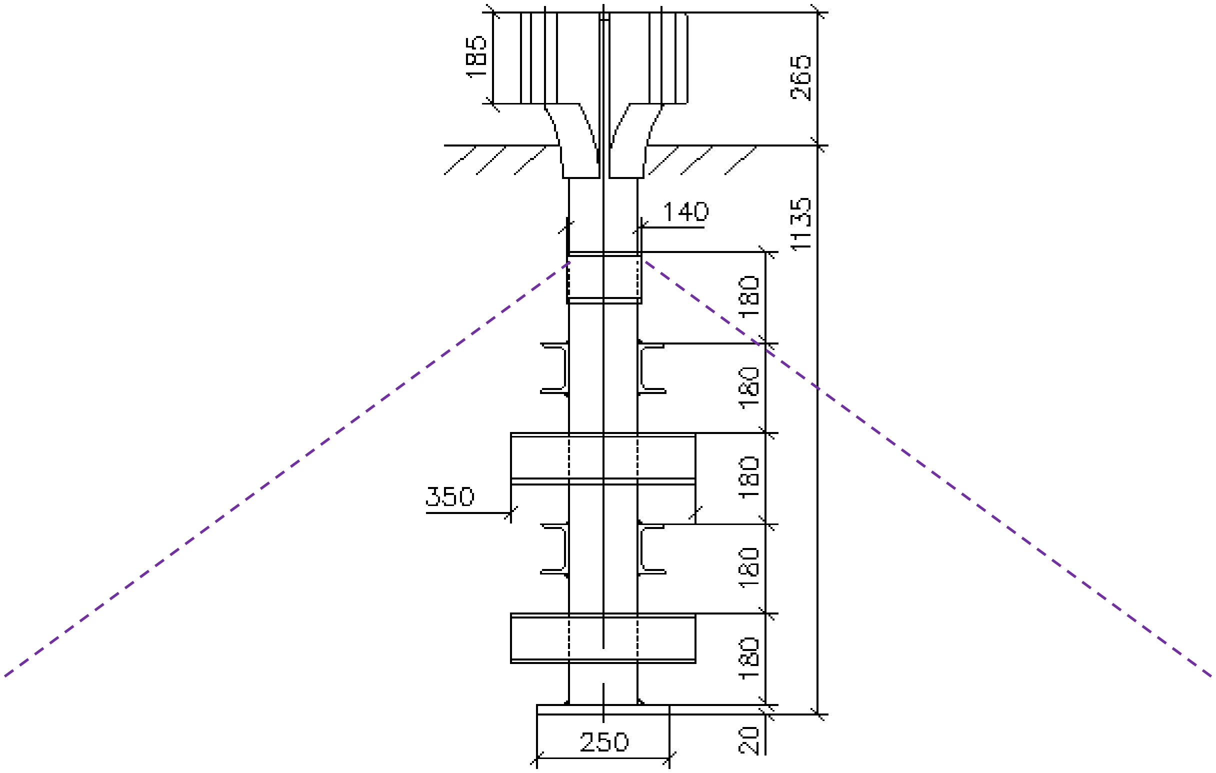


This is fundamentally a punching shear issue

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?